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Urban Runoff Characteristics

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~~This is an interim report on investigations for the development of a comprehensive storm water management model~~

Detailed information on the watershed characteristics and data on runoff quantity and quality have been compiled from a one year study of a combined sewer watershed of approximately 2380 acres in Cincinnati, Ohio. Collection of these data is planned to continue over a several year period. The information collected will be used to test and develop practical storm water management models.

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URBAN RUNOFF CHARACTERISTICS

BY

DIVISION OF WATER RESOURCES
DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY OF CINCINNATI

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This report has been reviewed by the Water Quality Office and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the Water Quality Office, nor does mention of trade names or commercial products constitute endorsement or recommendation for use.

ABSTRACT

This is an interim report on investigations for the refinement of the comprehensive Environmental Protection Agency, Water Quality Office, Storm Water Management Model.

Detailed information on the watershed characteristics and data on runoff quantity and quality have been compiled from a one year study of a combined sewer watershed of approximately 2380 acres in Cincinnati, Ohio. Collection of these data is planned to continue over a three year period.

The information collected will be used to test and refine the Storm Water Management Model, developed for the Environmental Protection Agency, Water Quality Office under contracts 14-12-501, 502, 503 with Metcalf & Eddy, Inc. et al (draft final report dated September 1970).

In addition, work has been done under this grant at the University of Cincinnati on urban runoff mathematical models. Information on these models is included in this report. It is intended that these models will be improved and verified as additional watershed data become available.

This interim report was submitted on the first year of investigations under research grant 11024 DQU between the Water Quality Office of the Environmental Protection Agency and the University of Cincinnati.

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42. Runoff Quantity and Quality Data - Bloody Run Sewer Watershed - Storm: September 25, 1970; Station LONGVIEW - 10' x 15' Sewer 173

43. Runoff Quantity and Quality Data - Bloody Run Sewer Watershed - Storm: September 25, 1970; Station OUTLET - 10' x 15' Sewer 175

44. Runoff Quantity and Quality Data - Bloody Run Sewer Watershed - Storm: September 26, 1970; Station BANK - 54" Sewer 178

45. Runoff Quantity and Quality Data - Bloody Run Sewer Watershed - Storm: September 26, 1970; Station BANK - 78" Sewer 180

46. Runoff Quantity and Quality Data - Bloody Run Sewer Watershed - Storm: September 26, 1970; Station LONGVIEW - 9' x 11' Sewer 182

47. Runoff Quantity and Quality Data - Bloody Run Sewer Watershed - Storm: September 26, 1970; Station LONGVIEW - 9' x 12' Sewer 184

48. Runoff Quantity and Quality Data - Bloody Run Sewer Watershed - Storm: September 26, 1970; Stations LONGVIEW (10' x 15') & OUTLET (10' x 15') 186

49. Runoff Quantity and Quality Data - Bloody Run Sewer Watershed - Storm: October 9 & 10, 1970; Station BANK - 54" Sewer 189

50. Runoff Quantity and Quality Data - Bloody Run Sewer Watershed - Storm: October 9 & 10, 1970; Station BANK - 78" Sewer.................... 191

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51. Runoff Quantity and Quality Data - Bloody Run Sewer Watershed - Storm: October 9 & 10, 1970; Station LONGVIEW - 11' x 9' Sewer 193

52. Runoff Quantity and Quality Data - Bloody Run Sewer Watershed - Storm: October 9 & 10, 1970; Station LONGVIEW - 12' x 9' Sewer 195

53. Runoff Quantity and Quality Data - Bloody Run Sewer Watershed - Storm: October 9 & 10, 1970; Station OUTLET - 10' x 15' Sewer 197

54. Runoff Quantity and Quality Data - Bloody Run Sewer Watershed - Storm: October 12 & 13, 1970; Station BANK - 54" and 78" Sewers 200

55. Runoff Quantity and Quality Data - Bloody Run Sewer Watershed - Storm: October 12 & 13, 1970; Station LONGVIEW - 11' x 9' & 12' x 9' Sewers 202

56. Runoff Quantity and Quality Data - Bloody Run Sewer Watershed - Storm: October 12 & 13, 1970; Station OUTLET - 10' x 15' Sewer 204

57. Runoff Quantity and Quality Data - Bloody Run Sewer Watershed - Storm: October 14, 1970; Station BANK - 54" and 78" Sewers 207

58. Runoff Quantity and Quality Data - Bloody Run Sewer Watershed - Storm: October 14, 1970; Station LONGVIEW - 11' x 9' & 12' x 9' Sewers 209

59. Runoff Quantity and Quality Data - Bloody Run Sewer Watershed - Storm: October 14, 1970; Station OUTLET - 10' x 15' Sewer 211

60. Runoff Quantity and Quality Data - Bloody Run Sewer Watershed - Storm: October 20, 1970; Station BANK - 54" & 78" Sewers 214

61. Runoff Quantity and Quality Data - Bloody Run Sewer Watershed - Storm: October 20, 1970; Station LONGVIEW - 11' x 9' Sewer 216

62. Runoff Quantity and Quality Data - Bloody Run Sewer Watershed - Storm: October 20, 1970; Station LONGVIEW - 12' x 9' Sewer 218

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63. Runoff Quantity and Quality Data - Bloody Run Sewer Watershed - Storm: October 20, 1970; Station OUTLET - 10' x 15' Sewer 220

64. Runoff Quantity and Quality Data - Bloody Run Sewer Watershed - Storm: October 20 (afternoon), 1970; Station OUTLET - 10' x 15' Sewer 223

65. San Francisco Laguna Street. March 10, 1967 Storm. Runoff Quality 327

66. San Francisco Laguna Street. March 15, 1967 Storm. Runoff Quality 329

67. Cincinnati Bloody Run Sewer Watershed. April 1, 1970 Storm. Runoff Quality 331

SECTION I

CONCLUSIONS & RECOMMENDATIONS

CONCLUSIONS

Automatic equipment for discharge measurements and sequential sampling of water quality has been installed for five sewer locations, including the outlet, in the combined sewer system of the Bloody Run Sewer Watershed in Cincinnati, Ohio.

Detailed physical information on the sewer system and watershed have been compiled along with runoff quantity and quality data beginning in the spring of 1970. The physiographic compilation includes parameters required for a storm water management model, i.e., imperviousness of surfaces, street cleaning operations, sewer system design, slopes of overland flow areas, usage of areas, storm inlets, curb and gutter design, etc. The runoff data include the following:

1. Discharge hydrograph measurements have been made at the five sewer locations for eighteen rain storms.

2. Runoff water quality samples have been collected on a sequential basis and analyzed for approximately eighteen rain storms at the five sewer locations. The analyses included BOD, COD, suspended solids (total and volatile), and chloride (for a portion of the samples).

3. Detailed measurements and analyses of the quantity and quality of dry weather flow were accomplished during the summer period.

Two mathematical models for simulation of urban runoff have been developed under this grant. They are presented in basic form in this report. One of the models is for simulation of the rate of water runoff and the other is for simulation of the rate of pollutional runoff. Examples for application of the water runoff hydrograph simulation model are provided with a complete computer program. These models will be improved and tested as additional data become available along with the Storm Water Management Model developed by Metcalf & Eddy, Inc., et al, under contracts 14-12-501, 502, 503, with the Environmental Protection Agency, Water Quality Office (draft final report dated September, 1970).

RECOMMENDATIONS

1. It is recommended that the collection of runoff quantity and quality data continue for the Bloody Run combined sewer watershed and that one additional station be added in the upper part of the watershed if possible.

2. It is recommended that a second watershed having a separate sewer system be instrumented and that data be collected similar to that from the Bloody Run combined sewer area. Plans are being prepared to carry this out for a watershed near the Bloody Run sewer area.

3. It is recommended that observations on the runoff of chloride concentrations be made during the winter period for the Bloody Run sewer area, so as to obtain information on pollution from street salting operations. A program for this sampling and analysis has been set up and is intended as a part of the 1971 winter operations under this grant.

SECTION II

INTRODUCTION

This is an interim report on the first year progress (October 1, 1969-September 30, 1970) under the research grant 11024 DQU to the University of Cincinnati by the Environmental Protection Agency, Water Quality Office.

The general objective of this project is to develop and verify a reliable method for control of the magnitude, frequency, and quality of combined sewer overflows from urban watersheds. The final result is expected to be a mathematical model and computer program verified for practical usage.

The main effort during the first year has been to establish a monitoring system and to gather as much field data as possible. In addition, considerable effort has been given to the development of mathematical models which will simulate and control a storm water overflow system. The monitoring system which has been established differs from various systems that have been set up for past studies in that it provides for observations of the rates of runoff (quantity and quality) at a number of locations within the sewer system as well as at the outlet of the drainage area. Many of the past studies have included observations only at the outlets.

The second year's program (October 1, 1970-September 30, 1971) will continue the gathering of field data to a maximum extent on the existing combined sewer watershed. It is also planned that a separate sewer drainage area, adjacent to the existing combined sewer watershed, will be set up with a monitoring system and that observations would begin there in the spring of 1971.

Testing and verification of the mathematical models will be carried out also during the second year with available data. These models will include the Storm Water Management Model furnished by EPA/WQO (developed by Metcalf and Eddy, Inc. and others under their contracts 14-12-501, 502, 503) and also those being developed by the University of Cincinnati as included in this report on a preliminary basis.

In testing and developing the models, study will be oriented toward the benefits which may be gained by the use of different forms of runoff storage at various points within a system during peak runoff periods.

Because of growing concern over the runoff of salt from streets during the winter period in northern urban areas, an ancillary program has been established to make chloride concentration observations at various points in the Bloody Run Sewer drainage system.

It is intended that the collection of samples and data would continue for at least a third year and that the overall operations would then be finally summarized for engineering application purposes.

SECTION III

DESCRIPTION OF THE BLOODY RUN SEWER WATERSHED

A combined sewer watershed of diversified usage in the City of Cincinnati was selected for the purpose of collecting data to develop and verify storm water management models for urban areas. The watershed is referred to as the Bloody Run Sewer System. It was so named because of an Indian massacre which took place in that area during an early historical period. Its location is in the northeast section of the City of Cincinnati as shown in Figure 1.

The area is comprised of 2,380 acres of rolling terrain. Topographically, the area is characterized by two main valleys running approximately east and west. Most of the commercial and industrial sections are located in these valleys; the residential housing is found on the ridges.

About 55% of the area is residential, 17% commercial, 5% industrial, and 22% open land and parks. A more detailed division of the Bloody Run Sewer Watershed into different land uses is presented in Figure 3 and Table 1.

The Bloody Run Sewer Watershed was divided into 37 subareas by using sewer, topographical, and zoning maps, with the concept that each area should include one major type of land use and should incorporate an individual inlet manhole. The resulting division is shown in Figure 4. The subareas have been numbered in such a way to follow the sewer network and facilitate the data tabulation for computer programming.

The physical characteristics of the subareas, such as the number of acres, length, average width which is used to calculate the distance that the overland runoff must travel before entering the inlet manhole, maximum and minimum elevation, average slope, and perviousness, are given in Tables 2 and 3.

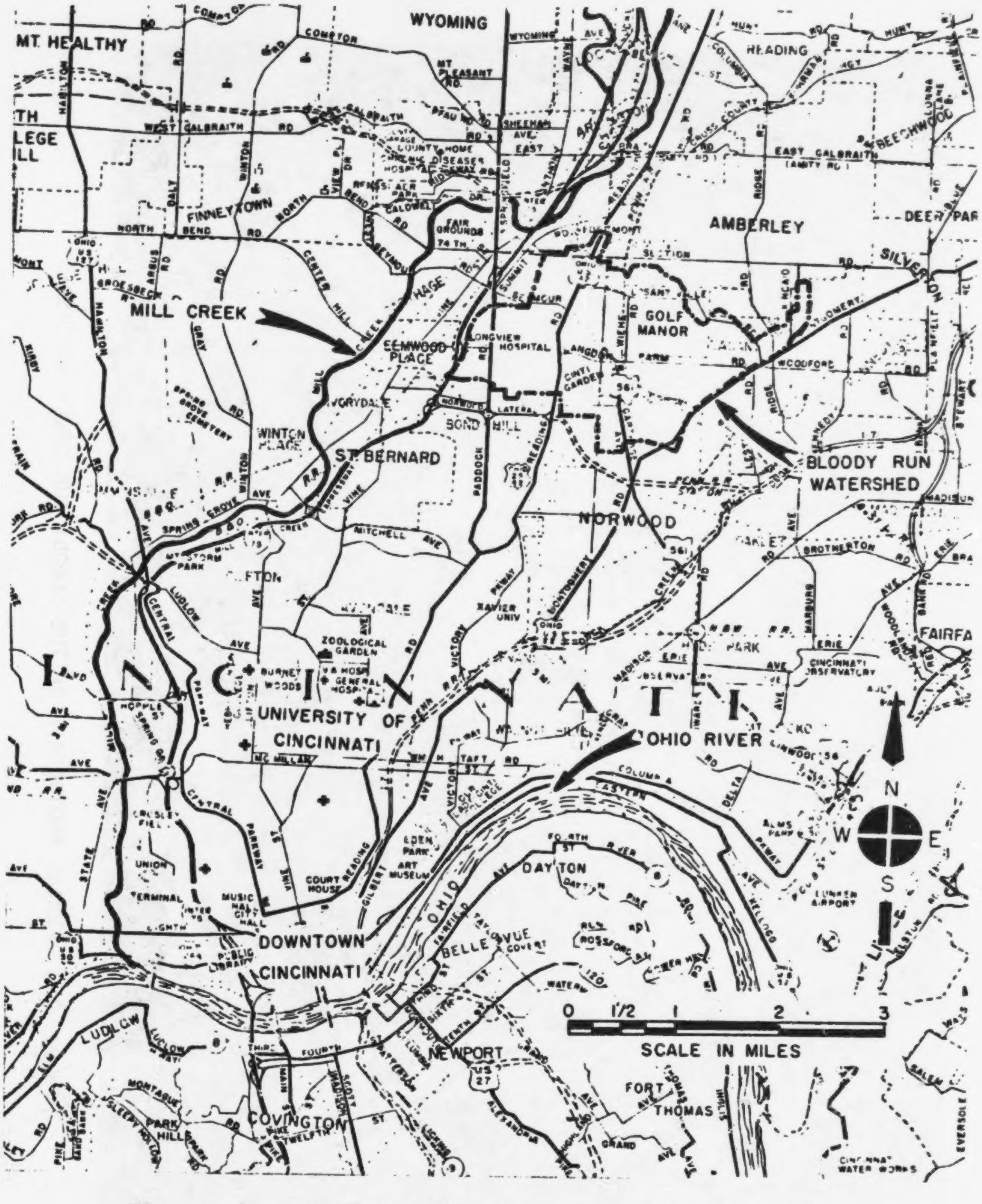
The total population according to the 1960 census is approximately 26,000 with an average of about 11 persons per acre. Figure 5 presents the 1960 census tracts covering the Bloody Run Sewer Watershed and Table 4 summarizes the data collected for these tracts. In Table 5, the same data have been extracted for each one of the 37 subareas which constitute the Bloody Run Sewer Watershed.

In order to calculate the overland flow from the street surfaces feeding the roadside gutters, the screed diagram of the streets according to their width is needed as given in Figures 6 and 7. For the estimation of the gutter flows, the details of rolling and battered curbs are given in Figure 8 together with two common sidewalk cross-sections.

Information on street cleaning operations, i.e. number of miles of streets swept and number of cubic yards of litter and debris removed, have been obtained from the City of Cincinnati Department of Public Works. These data are provided in Table 6 for each one of five areas which cover the Bloody Run Sewer Watershed as shown in Figure 9.

Infiltration tests are being performed at various locations of the Bloody Run Sewer Watershed in order to estimate the infiltration capacity curves for various types of soil and vegetative cover. The purpose of these tests is to obtain values for the maximum infiltration capacity rate, the minimum infiltration capacity rate, and the decay rate of infiltration. Tube flooding-type infiltrometers are being used. They consist of a metal tube 25" long with wall thickness 5/8" and inside diameter 8 7/8". To perform the infiltration test, the tube is jacked into the soil to a depth of about 21 inches. Water is then applied from graduated burettes to maintain a constant head of water sufficiently deep to submerge the vegetation. The burettes are read at successive time intervals to determine the rates and amounts of infiltration. A typical installation for infiltration test is shown in Figures 10 and 11.

The infiltration tests will be continued in the future under dry and wet antecedent conditions and with frost conditions until sufficient information is obtained to permit conclusions on the influence of vegetative cover, soil type, and temperature.


MILL CREEK
BLOODY RUN WATERSHED
OHIO RIVER
UNIVERSITY OF CINCINNATI
DOWNTOWN CINCINNATI
NORWOOD
ST. BERNARD
WYOMING
MT. HEALTHY
READING
AMBERLEY
GOLF MANOR
ELMWOOD PLACE
WINTON PLACE
CLIFTON
NEWPORT
COVINGTON
FORT THOMAS
BELLEVUE
DAYTON
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W
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1
2
3
SCALE IN MILES

Figure 1. GENERAL LOCATION OF THE BLOODY RUN SEWER DRAINAGE AREA IN CINCINNATI

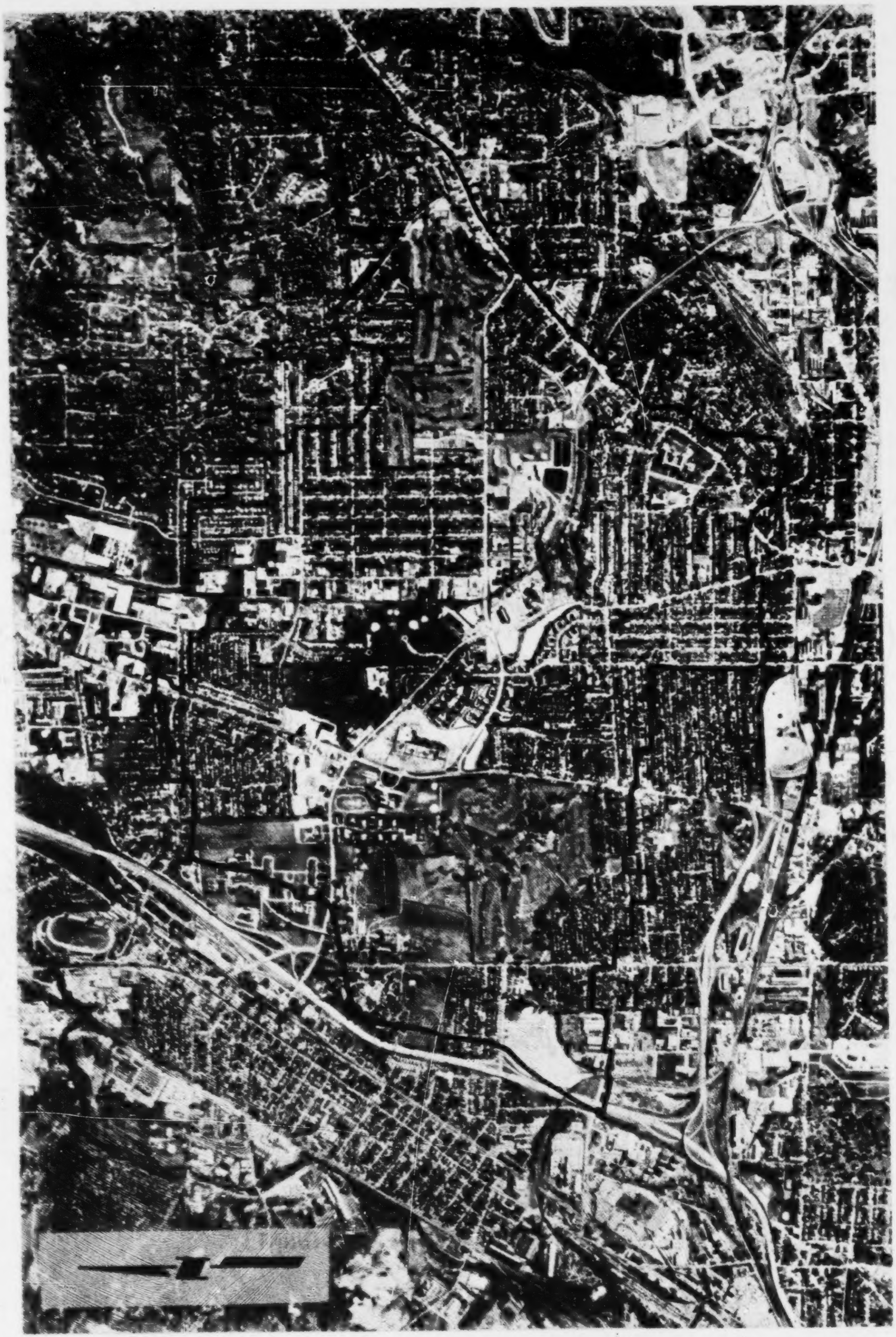


Figure 2 . AERIAL PHOTOGRAPH OF THE BLOODY RUN SEWER WATERSHED

NOT REPRODUCIBLE

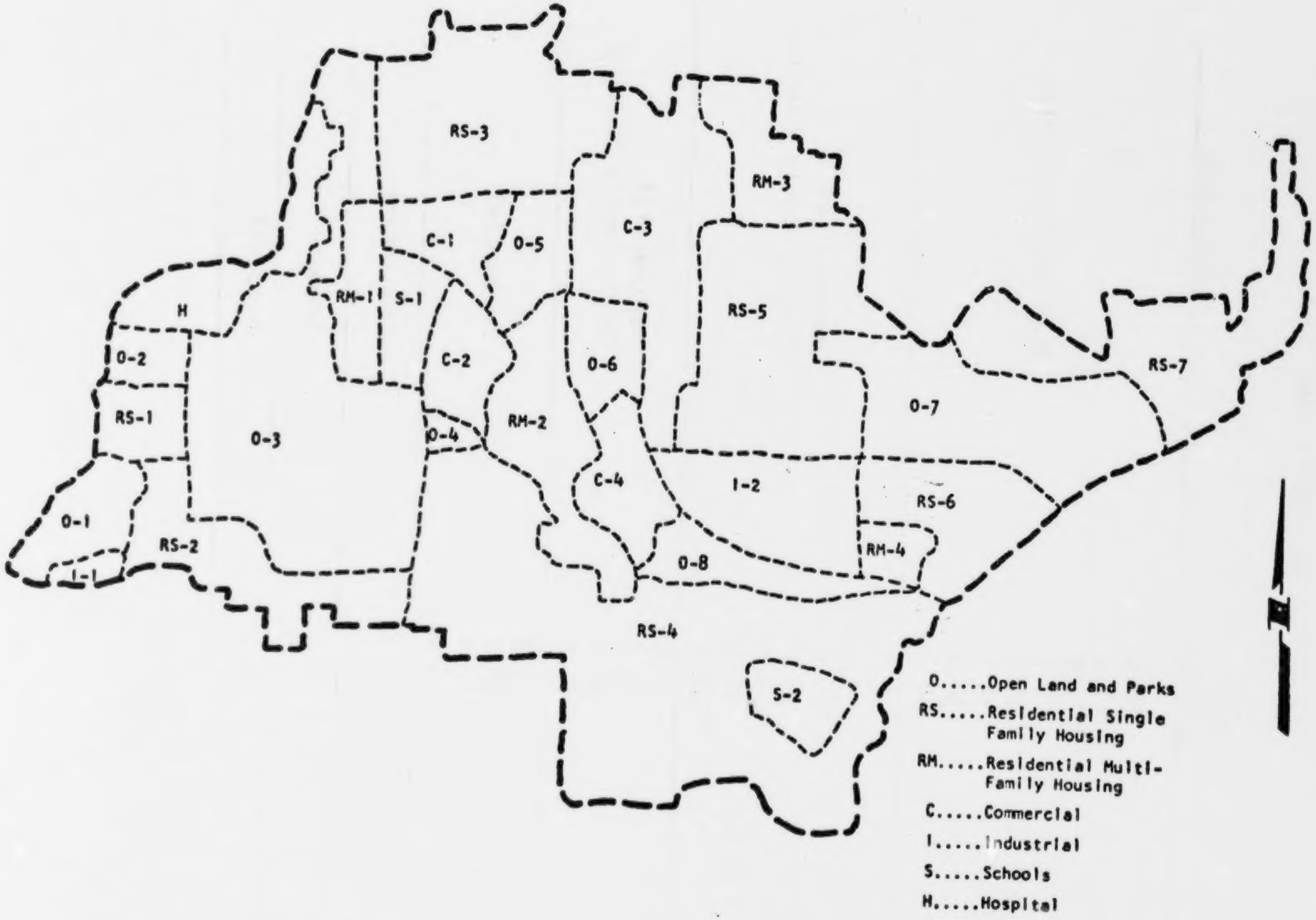
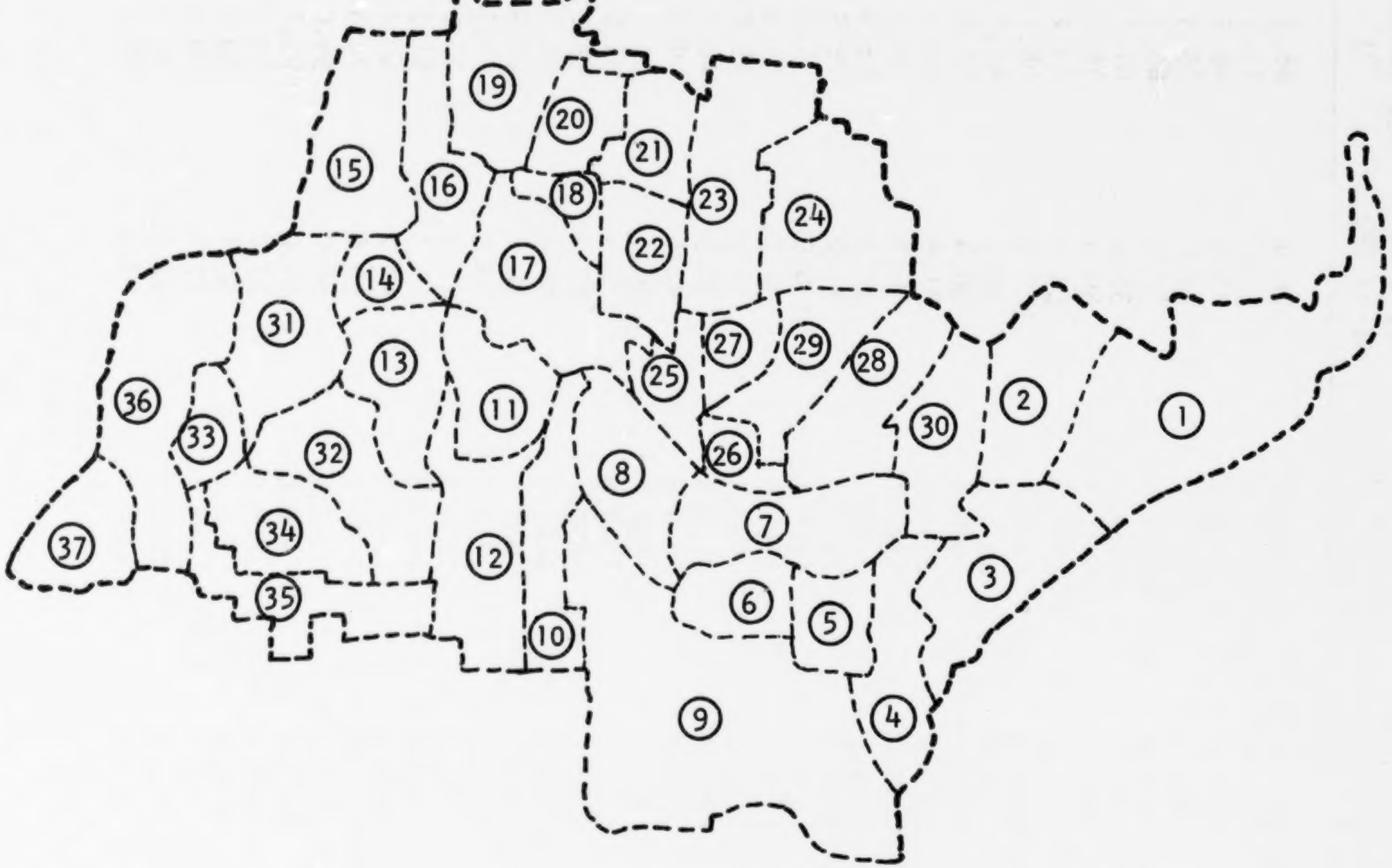

RS-3
RM-3
C-1
O-5
C-3
RM-1
S-1
H
RS-5
O-2
C-2
O-6
RS-7
RS-1
O-4
RM-2
O-7
O-3
C-4
I-2
RS-6
O-1
RS-2
RM-4
I-1
O-8
RS-4
S-2
O.....Open Land and Parks
RS.....Residential Single Family Housing
RM.....Residential Multi-Family Housing
C.....Commercial
I.....Industrial
S.....Schools
H.....Hospital

Figure 3. DIVISION OF BLOODY RUN SEWER WATERSHED INTO DIFFERENT LAND USES

Table 1. DIVISION OF BLOODY RUN SEWER WATERSHED INTO DIFFERENT LAND USES

Char.*	Area (acres)	Land Use
0-1	38.5	Open Land and Parks
0-2	18.8	
0-3	275.0	
0-4	9.2	
0-5	34.5	
0-6	34.5	
0-7	138.0	
0-8	32.2	
RS-1	27.6	Residential Single Family Housing
RS-2	113.0	
RS-3	160.0	
RS-4	396.0	
RS-5	216.0	
RS-6	78.0	
RS-7	137.5	
RM-1	27.6	Residential Multi-Family Housing
RM-2	85.5	
RM-3	57.5	
RM-4	11.5	
C-1	39.2	Commercial
C-2	38.0	
C-3	160.0	
C-4	46.0	
I-1	9.2	Industrial
I-2	79.5	
S-1	34.5	Schools
S-2	40.0	
H	39.0	Hospital

* referring to Figure 3.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37

Figure 4 . DIVISION OF BLOODY RUN SEWER BASIN INTO SUBAREAS

Table 2 . PHYSICAL CHARACTERISTICS OF THE BLOODY RUN SEWER WATERSHED

Subarea Number *	Length in Feet (L)	Average Width in Feet (A/L)	Area in Acres (A)	Perviousness (%)
1	5010.0	1530.0	176.0	66.3
2	2625.0	1138.4	68.6	71.3
3	2526.5	1262.1	73.2	46.5
4	625.1	4112.1	59.0	65.3
5	1062.5	1557.9	38.0	63.2
6	1500.0	969.9	33.4	61.1
7	3062.5	481.4	69.0	26.7
8	1375.0	1701.2	53.7	43.2
9	4562.5	2388.8	250.2	48.1
10	750.0	2625.2	45.2	41.5
11	1125.0	1486.8	38.4	18.6
12	1375.0	2591.4	81.8	45.5
13	1375.0	1522.7	49.8	89.2
14	2312.5	380.1	20.2	65.3
15	2687.5	1167.0	72.0	79.3
16	1437.5	5515.1	116.0	54.4
17	2500.0	1597.8	91.2	35.5
18	1125.0	693.1	17.9	75.5
19	1250.0	2052.5	58.9	60.2
20	1125.0	1645.6	42.5	60.3
21	875.0	1797.8	36.1	57.7
22	1125.0	1544.9	39.9	53.6
23	1062.5	3570.9	87.1	39.2
24	1562.5	2461.7	88.3	61.3
25	1562.5	839.1	30.1	55.4
26	1000.0	1310.0	29.9	28.3
27	750.0	1071.5	21.9	43.6
28	1125.0	2334.8	60.3	51.3
29	1000.0	2090.9	48.0	60.1
30	1125.0	1954.2	50.6	70.3
31	2312.5	1243.2	66.0	95.0
32	2600.0	900.0	54.0	95.1
33	1300.0	800.0	25.0	83.2
34	2625.0	896.0	50.1	89.1
35	3250.0	837.7	62.5	60.5
36	710.0	7600.0	122.0	50.3
37	875.0	1138.4	49.0	66.2

*referring to Figure 4 .

Table 3 . COMPUTED AVERAGE SLOPE OF SUBAREAS OF BLOODY RUN SEWER WATERSHED

Subarea Number *	Maximum Elevation	Minimum Elevation	Distance (feet)	Slope (%)
1	845	690	5000	3.1
2**	750	675	2400	3.1
	785	750	450	7.8
3**	700	645	1800	3.1
	730	675	700	7.8
4**	680	650	1300	2.3
	715	650	850	4.1
5	720	605	1050	11.2
6	685	625	600	10.5
7**	650	600	2300	2.2
	675	650	500	5.0
	675	650	400	6.3
8**	620	575	400	11.3
	620	575	600	7.5
9**	800	625	4000	4.4
	715	625	2600	3.4
10	625	560	2400	2.0
11	585	555	700	4.3
12	610	555	1700	3.2
13	595	550	900	5.0
14	580	545	1000	3.5
15**	600	560	1300	3.1
	590	560	700	4.3
16	595	540	1200	3.7
17	600	550	2100	2.3
18	590	550	1000	4.0
19	595	575	900	2.2
20	590	575	1000	1.5
21	575	555	600	4.2
22	600	575	1000	2.5
23	620	590	1400	5.7
24	725	605	1900	6.3
25	625	575	600	8.3
26	635	600	550	6.4
27	670	605	1100	5.9
28	725	640	2000	4.3
29	660	610	1400	3.6
30**	735	660	1400	5.4
	730	660	900	7.8
31**	590	540	1200	4.2
	580	540	700	5.7
32	600	535	2600	2.5
33	575	540	1300	2.7
34	575	535	1700	2.3
35	610	600	800	1.2
36**	585	530	750	7.3
	575	540	600	5.8
37	575	525	700	7.1

*referring to Figure 4.

**Subareas comprised of two or more smaller areas, each sloping in different directions.

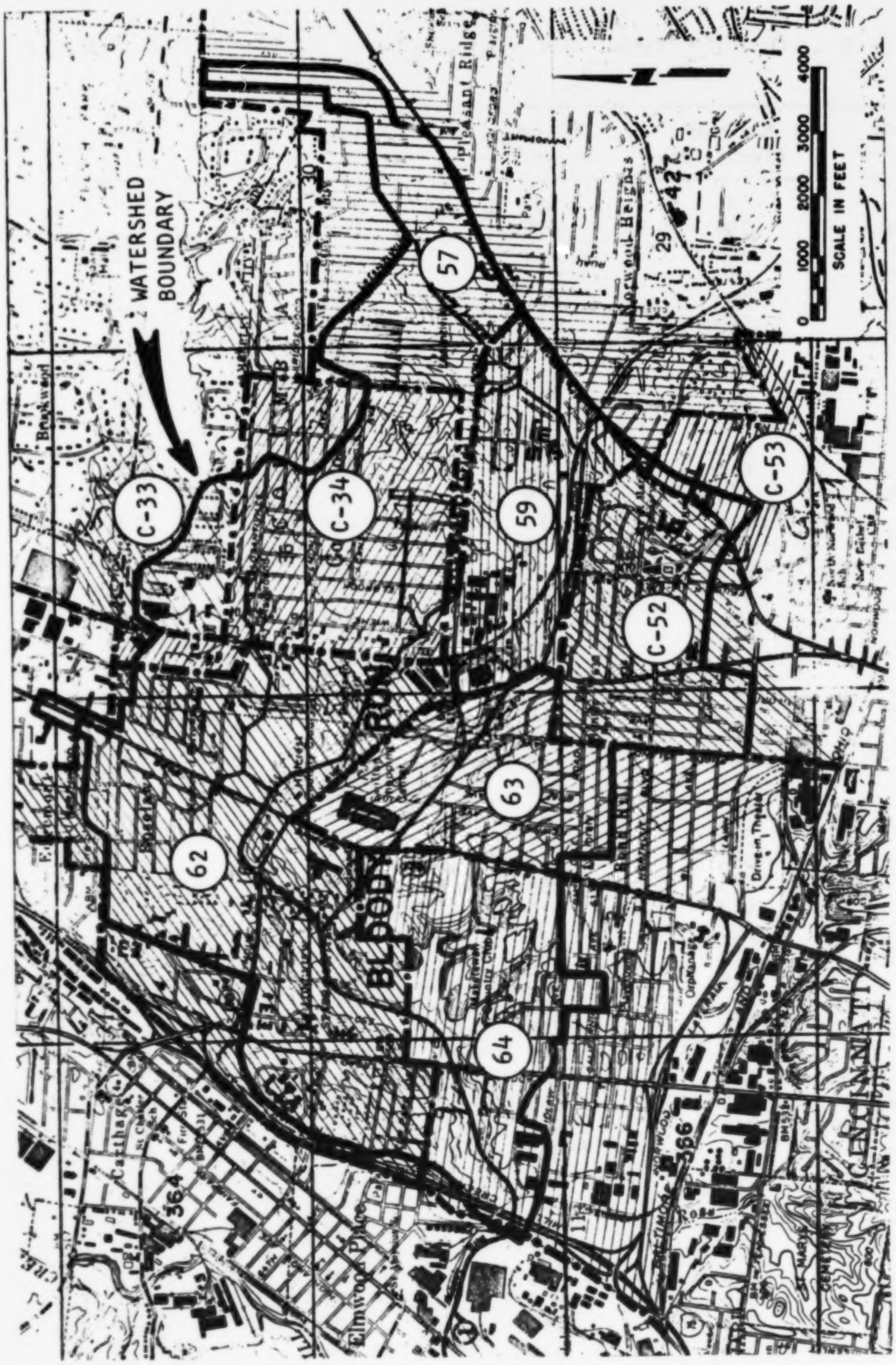

WATERSHED BOUNDARY
C-33
C-34
57
59
C-53
C-52
62
63
64
BLOODY RUN
Pleasant Ridge
Norwood Heights
Elmwood Place
0
1000
2000
3000
4000
SCALE IN FEET

Figure 5 . CENSUS TRACTS 1960 COVERING BLOODY RUN SEWER WATERSHED

TABLE 4 - CENSUS DATA, YEAR 1960, FOR BLOODY RUN SEWER WATERSHED

(Tract Boundaries on Tract Division Map)

Census Tract*	Total No. Persons per Acre	Total No. Dwelling Units	No. People Living in Avg. Dwelling Unit	Median Family Income in $1,000	No. Owner Occupied Houses	Avg. Market Value Owner Occupied Houses in $1,000
C34	11.2585	1537	3.0241	7.53	467	17.30
C33	1.4589	1038	3.3824	14.726	1024	35.00
57	10.1005	3249	2.7452	7.659	1738	17.40
59	8.3034	617	2.7780	7.640	308	19.00
C52**	19.3363	--	--	--	--	--
63	17.0912	2794	2.6306	7.388	1031	17.30
64	5.3840	1499	2.8833	7.243	683	17.00
62	10.4767	2002	4,5310	8.607	795	22.70

*Referring to Figure 5.

**No detailed census data available from the City of Norwood.

TABLE 5 - CENSUS DATA, YEAR 1960, FOR BLOODY RUN SEWER WATERSHED

(Sub-area Boundaries on Sub-Area Division Map)

Sub-Area* #	Total # Persons per Acre	Total # Dwelling Units	# People Living in Avg. Dwelling Unit	Median Family Income ($1000)	Avg. Market Value Owner Occupied Units ($1000)
1	9.29	608	2.83	8.34	21.31
2	9.72	230	2.65	7.62	17.42
3	8.72	215	2.79	7.64	19.00
4	13.46	178	2.78	7.64	19.00
5	15.19	111	2.78	7.64	19.00
6	13.97	98	2.78	7.64	19.00
7	8.30	203	2.78	7.64	19.00
8	14.68	302	2.89	7.60	18.20
9	18.19	1620	2.63	7.38	17.30
10	17.09	307	2.63	7.38	17.30
11	16.74	223	2.75	7.47	17.38
12	12.57	395	2.71	7.29	17.08
13	8.39	105	3.91	8.34	20.91
14	10.48	40	4.53	8.61	22.70
15	10.48	140	4.53	8.61	22.70
16	10.48	100	4.53	8.61	22.70
17	12.54	335	3.90	8.16	20.85
18	10.48	40	4.53	8.61	22.70
19	10.48	140	4.53	8.61	22.70
20	5.86	130	4.53	8.61	22.70
21	10.48	50	4.14	12.72	31.30
22	6.67	110	4.53	8.61	22.70
23	6.73	192	3.26	11.21	26.32
24	10.98	321	3.12	8.33	19.23
25	11.01	91	3.89	8.23	20.49
26	9.22	58	2.79	7.39	17.81
27	11.26	81	3.02	7.53	17.30
28	11.62	231	3.15	7.55	17.43
29	11.26	179	3.02	7.53	17.30
30	13.09	57	3.70	7.58	18.54
31	10.48	124	4.53	8.61	22.70
32	5.38	100	2.88	7.24	17.00
33	7.93	47	3.37	7.21	18.07
34	5.38	93	2.88	7.24	17.00
35	5.38	116	2.88	7.24	17.00
36	8.38	266	3.99	8.24	20.95
37	10.48	113	4.53	8.61	22.70

*referring to Figure 4.

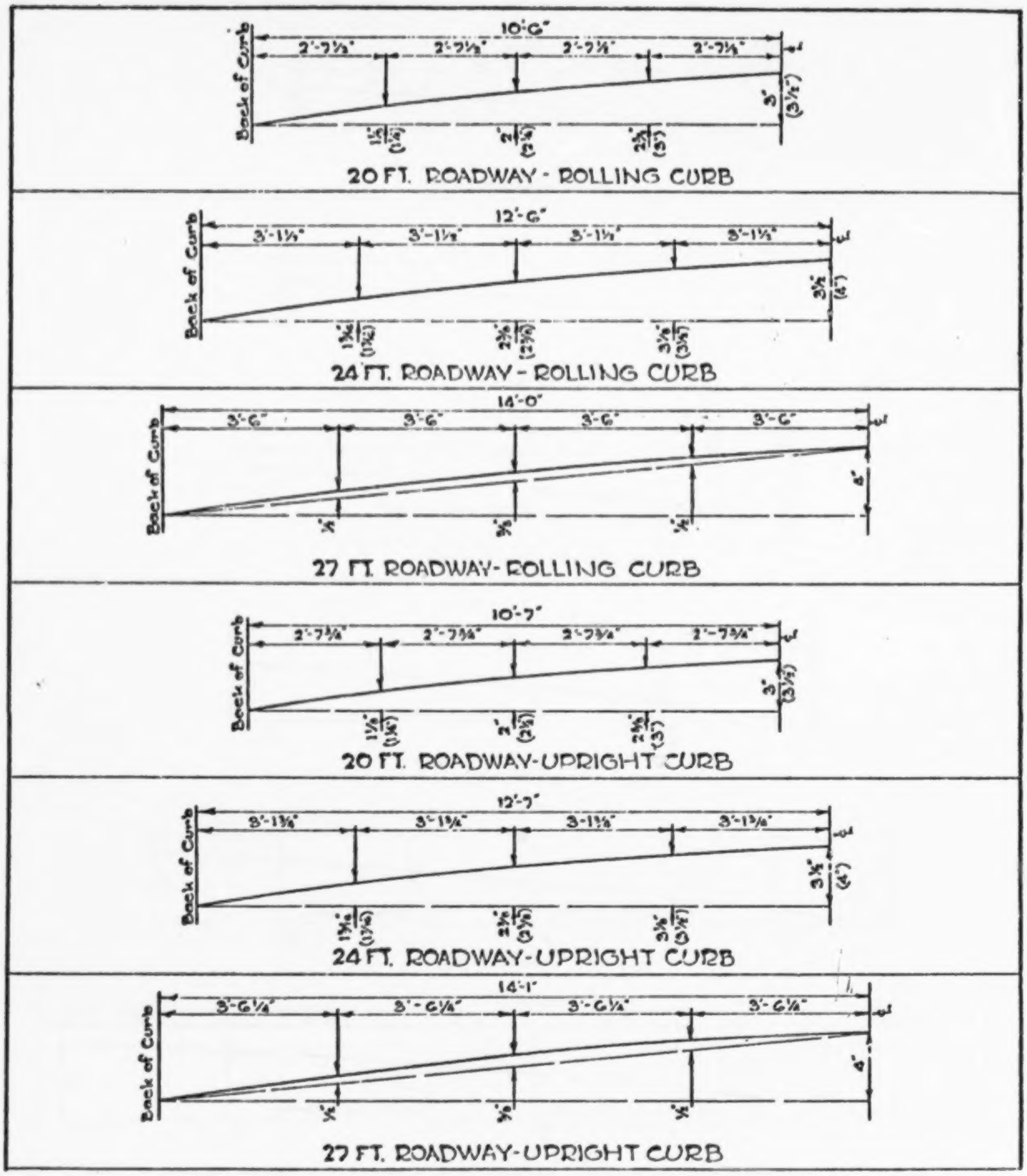
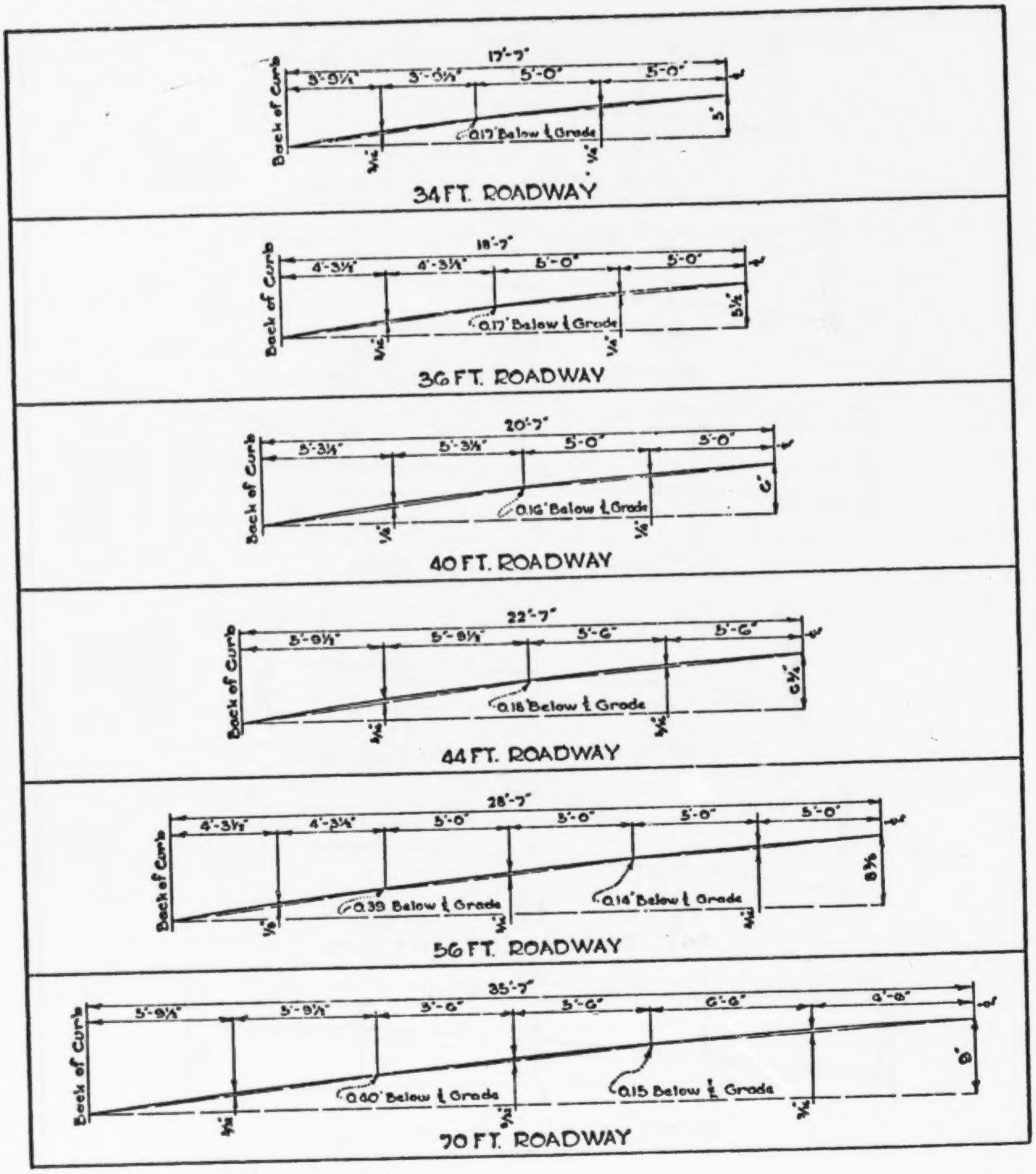
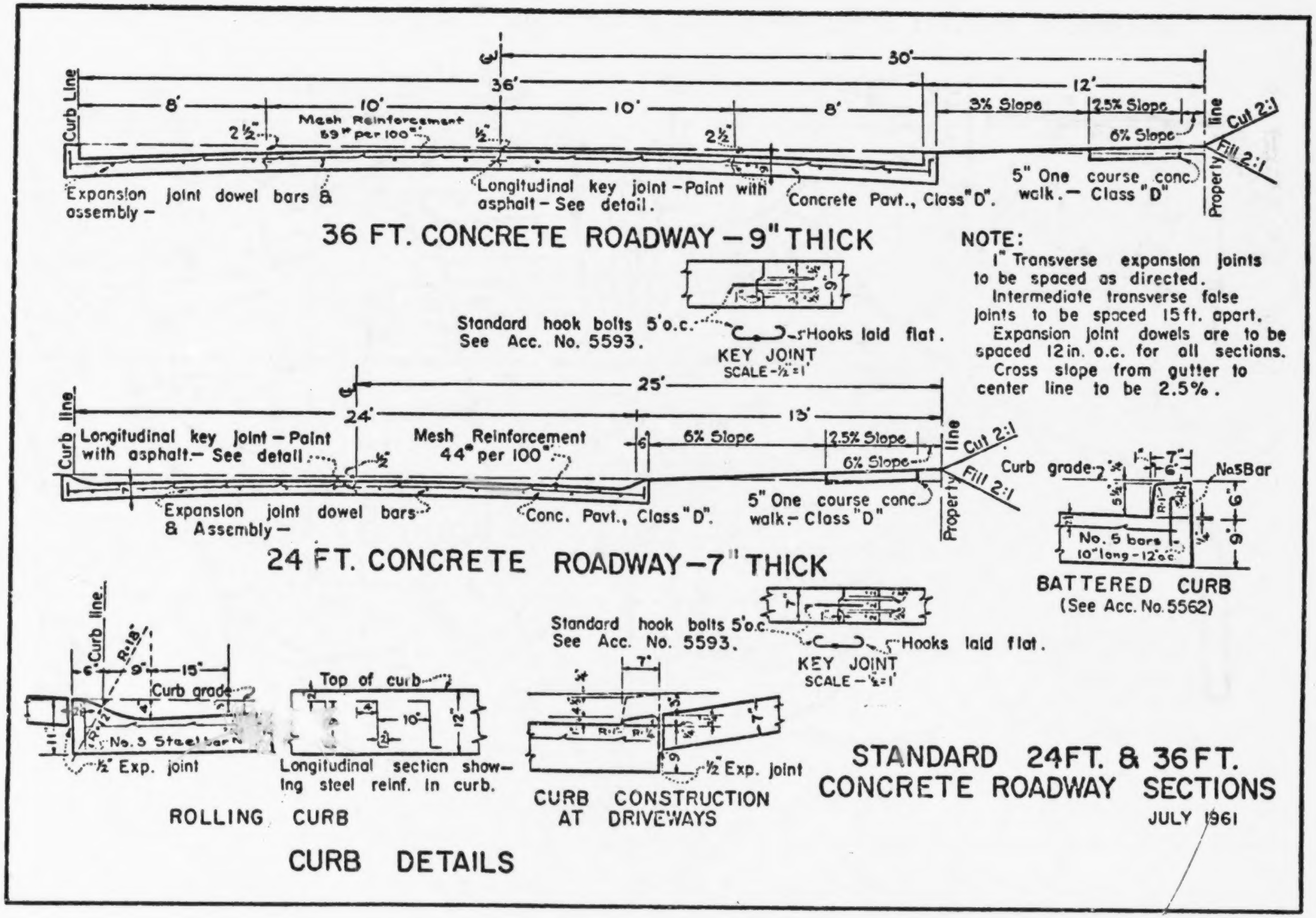

Back of Curb
10'-6"
2'-7½"
2'-7½"
2'-7½"
2'-7½"
3"
(3½")
20 FT. ROADWAY - ROLLING CURB
Back of Curb
12'-6"
3'-1½"
3'-1½"
3'-1½"
3'-1½"
(4")
24 FT. ROADWAY - ROLLING CURB
Back of Curb
14'-0"
3'-6"
3'-6"
3'-6"
3'-6"
4"
27 FT. ROADWAY - ROLLING CURB
Back of Curb
10'-7"
2'-7¾"
2'-7¾"
2'-7¾"
2'-7¾"
3"
(3½")
2"
(3")
20 FT. ROADWAY-UPRIGHT CURB
Back of Curb
12'-7"
3'-1¾"
3'-1¾"
3'-1¾"
3'-1¾"
(4")
24 FT. ROADWAY-UPRIGHT CURB
Back of Curb
14'-1"
3'-6¼"
3'-6¼"
3'-6¼"
3'-6¼"
4"
27 FT. ROADWAY-UPRIGHT CURB

Figure 6 . SCREED DIAGRAMS FOR 20-24 & 27 FOOT ROADWAYS


17'-7"
3'-9½"
3'-9½"
5'-0"
5'-0"
Back of Curb
0.17' Below ℄ Grade
5"
34 FT. ROADWAY
18'-7"
4'-3½"
4'-3½"
5'-0"
5'-0"
Back of Curb
0.17' Below ℄ Grade
5½"
36 FT. ROADWAY
20'-7"
5'-3½"
5'-3½"
5'-0"
5'-0"
Back of Curb
0.16' Below ℄ Grade
6"
40 FT. ROADWAY
22'-7"
5'-9½"
5'-9½"
5'-6"
5'-6"
Back of Curb
0.18' Below ℄ Grade
6½"
44 FT. ROADWAY
28'-7"
4'-3½"
4'-3½"
5'-0"
5'-0"
5'-0"
5'-0"
Back of Curb
0.39 Below ℄ Grade
0.14' Below ℄ Grade
8½
56 FT. ROADWAY
35'-7"
5'-9½"
5'-9½"
5'-6"
5'-6"
6'-6"
6'-6"
Back of Curb
0.40' Below ℄ Grade
0.15 Below ℄ Grade
10"
70 FT. ROADWAY

NOTE: Roadway widths are based
on the use of 7" wide Integral
Conc. Curb.

Figure 7 . SCREED DIAGRAMS FOR 34, 36, 40, 44, 56 & 70 FOOT ROADWAYS


Curb Line
36'
30'
8'
10'
10'
8'
12'
2½"
Mesh Reinforcement 59# per 100'
½"
2½"
3% Slope
2.5% Slope
6% Slope
Property line
Cut 2:1
Fill 2:1
Expansion joint dowel bars & assembly –
Longitudinal key joint – Paint with asphalt – See detail.
Concrete Pavt., Class "D".
5" One course conc. walk. – Class "D"
36 FT. CONCRETE ROADWAY – 9" THICK
Standard hook bolts 5' o.c. See Acc. No. 5593.
Hooks laid flat.
KEY JOINT
SCALE – ½" = 1'
Curb line
24'
25'
13'
Longitudinal key joint – Paint with asphalt – See detail
½"
Mesh Reinforcement 44# per 100'
6"
6% Slope
2.5% Slope
6% Slope
Property line
Cut 2:1
Fill 2:1
Expansion joint dowel bars & Assembly –
Conc. Pavt., Class "D".
5" One course conc. walk. – Class "D"
24 FT. CONCRETE ROADWAY – 7" THICK
Curb grade
No. 5 Bar
No. 5 bars 10" long – 12" o.c.
BATTERED CURB
(See Acc. No. 5562)
Standard hook bolts 5' o.c. See Acc. No. 5593.
Hooks laid flat.
KEY JOINT
SCALE – ½" = 1'
Curb line.
6"
9"
15"
R=18"
Curb grade
No. 3 Steel bar
½" Exp. joint
ROLLING CURB
Top of curb
10"
12"
Longitudinal section showing steel reinf. in curb.
7"
½" Exp. joint
CURB CONSTRUCTION AT DRIVEWAYS
CURB DETAILS
STANDARD 24 FT. & 36 FT. CONCRETE ROADWAY SECTIONS
JULY 1961

NOTE:
1" Transverse expansion joints to be spaced as directed.
Intermediate transverse false joints to be spaced 15 ft. apart.
Expansion joint dowels are to be spaced 12 in. o.c. for all sections.
Cross slope from gutter to center line to be 2.5%.

Figure 8 . STANDARD 24 FOOT & 36 FOOT CONCRETE ROADWAY SECTIONS

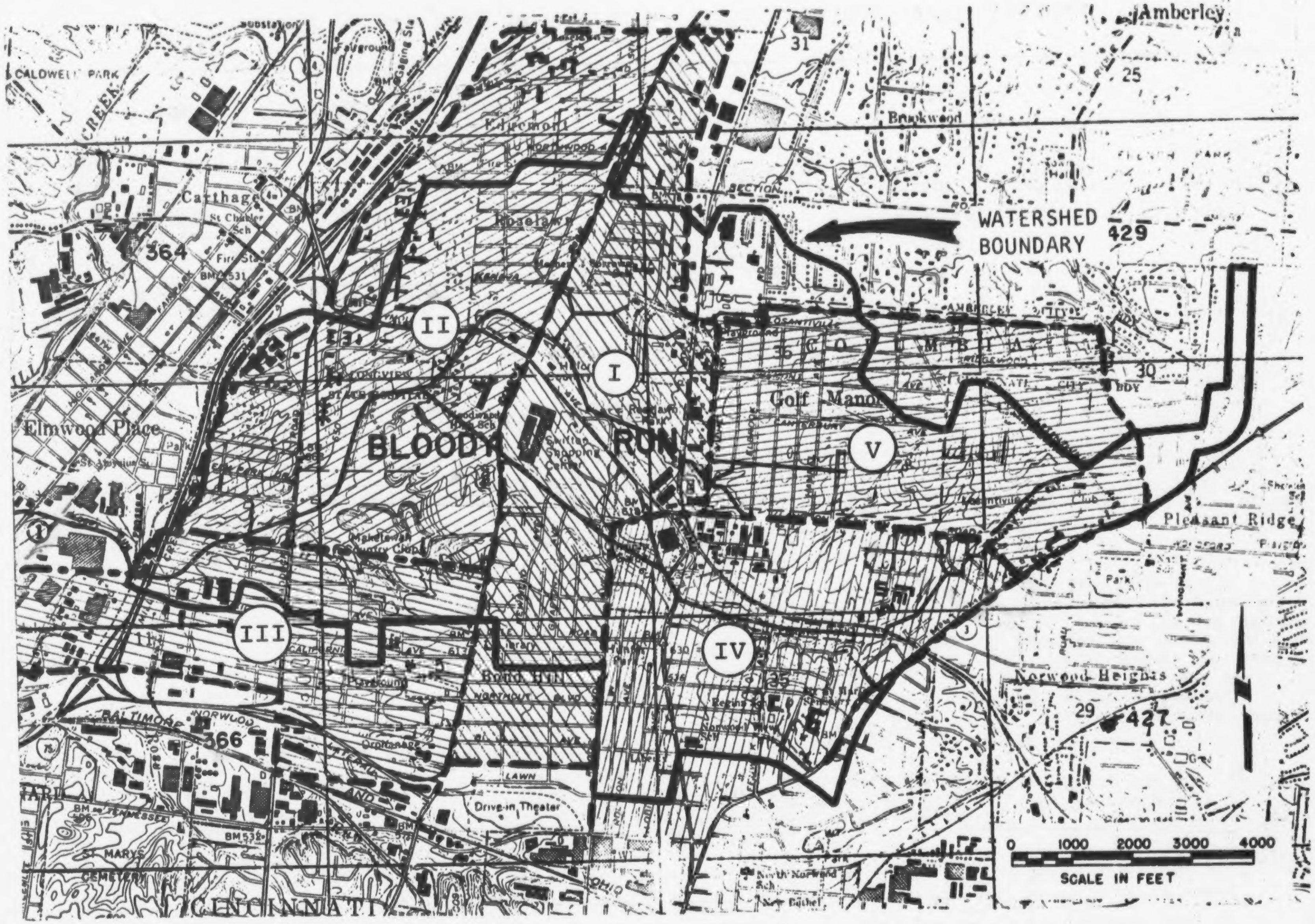

WATERSHED BOUNDARY
429
I
II
III
IV
V
BLOODY RUN
Amberley
Brookwood
Carthage
Elmwood Place
Golf Manor
Roselawn
Bond Hill
Pleasant Ridge
Norwood Heights
CALDWELL PARK
CINCINNATI
364
366
427
0 1000 2000 3000 4000
SCALE IN FEET

Figure 9. DIVISION OF BLOODY RUN SEWER WATERSHED FOR STREET CLEANING INFORMATION

Table 6 . STREET CLEANING OF THE BLOODY RUN SEWER WATERSHED
(data from City of Cincinnati)

AREA I*

Date	Miles of Streets Swept	Litter-Debris Removed	
		Cubic Yards	Pounds**
3/ 2/70	average 10.7 miles/run	7.3	17500
3/16/70		6.4	15500
3/31/70		7.1	17000
4/13/70		7.8	19000
4/26/70		3.9	10000
5/11/70		8.2	20000
5/25/70		8.3	20000
6/ 9/70		9.8	24000
6/22/70		7.2	17000
7/ 6/70		7.0	17000
7/20/70	10.0	8.5	21000
8/10/70	7.5	5.0	12000
8/31/70	8.5	5.0	12000
9/21/70	8.0	6.0	15000
10/12/70	6.0	5.0	12000
11/ 2/70	10.0	30.0^{+}	unknown

* referring to Figure 9 .

** one cubic yard has an average weight from 2200 to 2400 lbs.

+ Leaf removal operations. Normal street sweepings and leaves mixed.

Table 6 . STREET CLEANING OF THE BLOODY RUN SEWER WATERSHED
(data from City of Cincinnati)

AREA II*

Date	Miles of Streets Swept	Litter-Debris Removed	
		Cubic Yards	Pounds**
3/ 9/70	← average 8.1 miles/run →	7.1	17000
3/23/70		7.4	18000
4/ 7/70		5.1	12000
4/20/70		6.8	16000
5/ 5/70		6.7	16000
5/18/70		6.2	15000
6/ 1/70		7.5	18000
6/15/70		6.3	15000
6/30/70		6.6	16000
7/13/70		6.0	14500
7/27/70	14.0	10.0	24000
8/17/70	17.0	10.0	24000
9/ 8/70	17.0	10.0	24000
9/28/70	17.0	15.0	36000
10/19/70	10.8	15.0	36000
11/ 9/70	18.0	100.0^+	unknown

---

* referring to Figure 9 .

** one cubic yard has an average weight from 2200 to 2400 lbs.

+ Leaf removal operations. Normal street sweepings and leaves mixed.

Table 6 . STREET CLEANING OF THE BLOODY RUN SEWER WATERSHED
(data from City of Cincinnati)

AREA III*

Date	Miles of Streets Swept	Litter-Debris Removed	
		Cubic Yards	Pounds**
3/16/70	average 9.7 miles/run	8.1	19500
3/30/70		8.6	20500
4/13/70		9.6	23000
4/28/70		5.7	14000
5/11/70		8.9	21500
5/25/70		9.9	24000
6/ 8/70		12.7	30500
6/29/70		8.5	20500
8/ 3/70	20.0	10.0	24000
8/24/70	18.0	15.0	36000
9/14/70	14.0	10.0	24000
10/ 5/70	18.0	10.0	24000
10/26/70	18.0	25.0+	unknown
11/16/70	10.0	60.0+	unknown

* referring to Figure 9 .

** one cubic yard has an average weight from 2200 to 2400 lbs.

+ Leaf removal operations. Normal street sweepings and leaves mixed.

Table 6 . STREET CLEANING OF THE BLOODY RUN SEWER WATERSHED
(data from City of Cincinnati)

AREA IV*

Date	Miles of Street Swept	Litter-Debris Removed	
		Cubic Yards	Pounds**
3/12/70	← average 8.1 miles/run →	6.3	14000
3/26/70		5.8	13000
4/ 9/70		6.6	14500
4/23/70		5.2	11500
5/ 7/70		4.1	9000
5/21/70		6.9	15000
6/ 4/70		6.3	14000
6/18/70		5.1	11000
7/20/70	8.0	6.5	14500
8/10/70	7.5	5.0	11000
9/31/70	8.5	5.0	11000
9/21/70	8.0	6.0	13000
10/12/70	6.0	5.0	11000
11/ 2/70	8.0	26.0^+	unknown

* referring to Figure 9 .

** one cubic yard has an average weight from 2200 to 2400 lbs.

+ Leaf removal operations. Normal street sweepings and leaves mixed.

Table 6 . STREET CLEANING OF THE BLOODY RUN SEWER WATERSHED
(data from City of Cincinnati)

AREA V*

Date	Miles of Streets Swept	Litter-Debris Removed	
		Cubic Yards	Pounds**
3/ 6/70	average 6.7 miles/run	4.3	10000
3/20/70		4.1	9500
4/ 4/70		3.6	8300
4/17/70		3.7	8500
5/ 1/70		2.1	5000
5/15/70		2.9	7000
5/29/70		3.9	9000
6/12/70++		4.3	10000

* referring to Figure 9 .

** one cubic yard has an average weight from 2200 to 2400 lbs.

++ no data available after this date

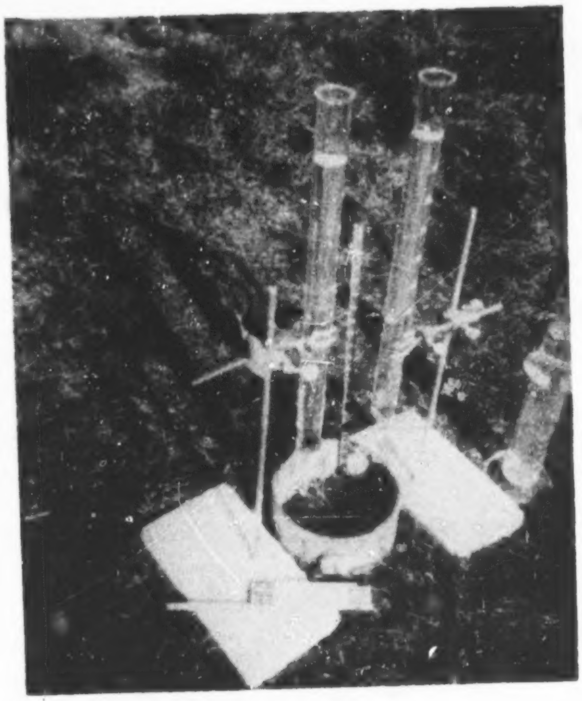


Figure 10 .

FLOODING-TYPE TUBE INFILTROMETER

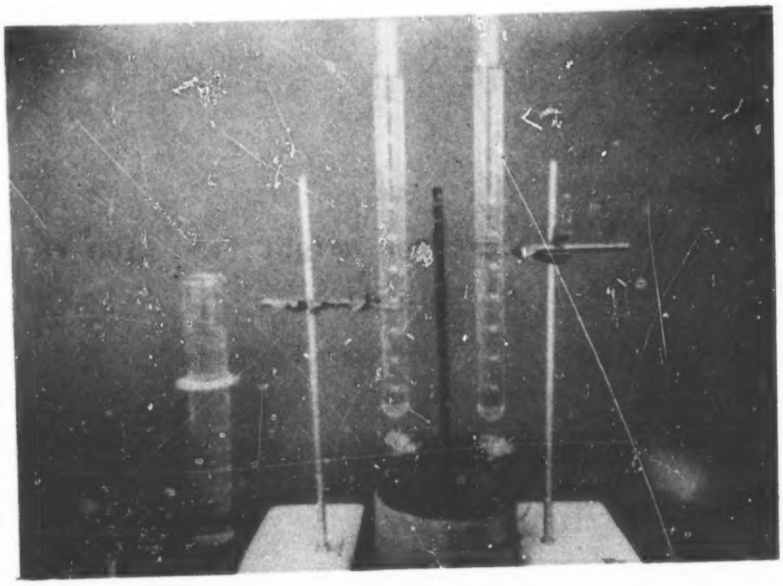


Figure 11 .

FLOODING-TYPE TUBE INFILTROMETER

SECTION IV

DESCRIPTION AND LAYOUT OF THE SEWER AND MONITORING SYSTEMS

The Bloody Run Sewer network is a combined system with a main trunk line that splits into three branches following the valleys of the watershed. Figure 12 presents the general sewer layout with the dimensions of the bigger than 27 inches pipes.

The sewer cutfall is located at the southwestern tip of the watershed and discharges into an interceptor leading to the Mill Creek Waste Water Treatment Plant. Overflows from storms are discharged directly into the Mill Creek through an open channel.

In Figure 13, the elements of the sewer system (pipes and manholes) are numbered for identification. Manholes are located wherever there is a significant change in pipe size, direction, or slope. Each of the 37 subareas, previously described, has its individual inlet manhole. The size, slope, and length of the sewer pipes is tabulated in Table 7.

Other elements of the Bloody Run Sewer System such as standard gutter inlets, standard valley inlets, standard ditch inlets, standard manholes for all the pipe sizes, and standard drop manholes are presented in Figures 14 to 28.

Three raingages manufactured by the Belfort Instrument Company, record automatically the accumulative mass curve of each rainfall. The raingages (Figure 30) record the rainfall by use of a weighing mechanism which causes a pen to trace changes on a chart in a pre-balanced collection system. The charts, which can be of 24 or 48 hours, record up to 6 inches of total rainfall. The location of the three raingages is shown on Figure 29. The first raingage is located immediately southwest of the study area on the roof of the Environmental Protection Agency, Air Pollution Control Office at 1055 Laidlaw Avenue. The rotation period of this raingage is 48 hours. The second raingage is located immediately southeast of the study area on the roof of the Environmental Protection Agency, Solid Wastes Office and Water Quality Office at 5555 Ridge Avenue. This raingage has a rotation period of 48 hours. The third raingage is installed in the Bloody Run Sewer Basin, on the roof of Woodward High School, at Reading Road in front of Swifton Shopping Center. This raingage has a rotation period of 24 hours.

Three monitoring stations are taking flow measurements and samples automatically during each rainfall. The locations of these stations are shown on Figure 29.

The "Bank" monitoring station is located off the rear of the parking lot of the Central Trust Bank near Swifton Shopping Center at the corner of Reading Road and Losantiville, as shown on Figure 31. The sampling and flow measuring apparatus (Figures 32, 33, and 34) are connected with a 54" and a 78" sewer which join there (Figure 35). These two pipes have been photographed during dry weather flow and are presented in Figures 36 and 37.

The "Longview" monitoring station is located at the back yard of the Longview Hospital on the corner of Paddock Road and Seymour Avenue as shown on Figures 38 and 39. The sampling and flow measuring apparatus shown on Figures 40 and 41 are connected with a 11' x 9' and a 12' x 9' semi-circular sewer which join there.

The "Outlet" monitoring station is located 50 yards from the intersection of Murray and Prosser Streets, on Prosser Street as shown on Figures 42 and 43. The gate covering the storm water outfall is shown on Figure 44. Automatic equipment has been installed (Figures 45, 46, and 47) which takes samples and flow measurements from the 10' x 15' sewer, 20 feet upstream from the interceptor.

The flow measuring apparatus consists of a compressor, a manometer, and a Taylor pressure type recorder. This recorder operates by measuring the pressure due to the depth and velocity of water flowing into the pipe by bubbling air through a long tube inserted into the water. The gage releases air at a constant pressure and as the depth and the velocity of flow changes, the pressure differential is recorded on a circular chart in inches of water. This pressure differential is actually the specific energy which is equal to the sum of the pressure head (depth of flow y) and the velocity head ($V^2/2g$). To obtain readily the discharge corresponding to the measured specific energy, Manning's equation has been used to express the depth and the velocity of flow as functions of the discharge and the hydraulic radius of the sewer cross sections. Thus the curves on Figures 48, 49, 51, 53 and 55 have been calculated relating the measured specific energy and the corresponding discharge at the five monitored locations.

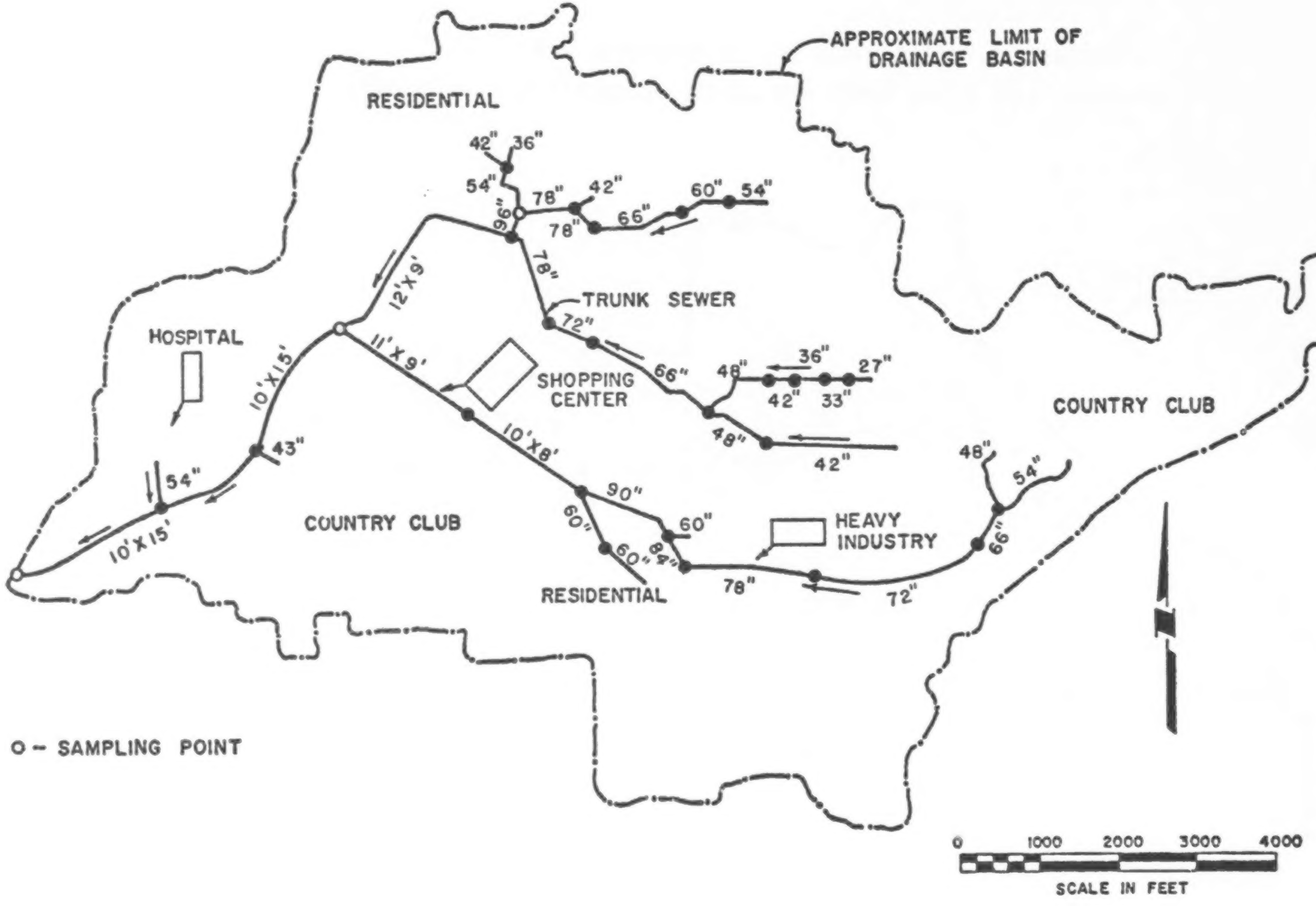

APPROXIMATE LIMIT OF DRAINAGE BASIN
RESIDENTIAL
42"
36"
54"
96"
78"
42"
78"
66"
60"
54"
78"
TRUNK SEWER
72"
66"
48"
36"
27"
42"
33"
48"
42"
HOSPITAL
12'X9'
11'X9'
10'X15'
SHOPPING CENTER
10'X8'
43"
54"
10'X15'
COUNTRY CLUB
90"
60"
60"
60"
84"
RESIDENTIAL
78"
72"
HEAVY INDUSTRY
66"
48"
54"
COUNTRY CLUB
O – SAMPLING POINT
0
1000
2000
3000
4000
SCALE IN FEET

Figure 12. GENERAL SEWER LAYOUT MAP OF BLOODY RUN SEWER BASIN

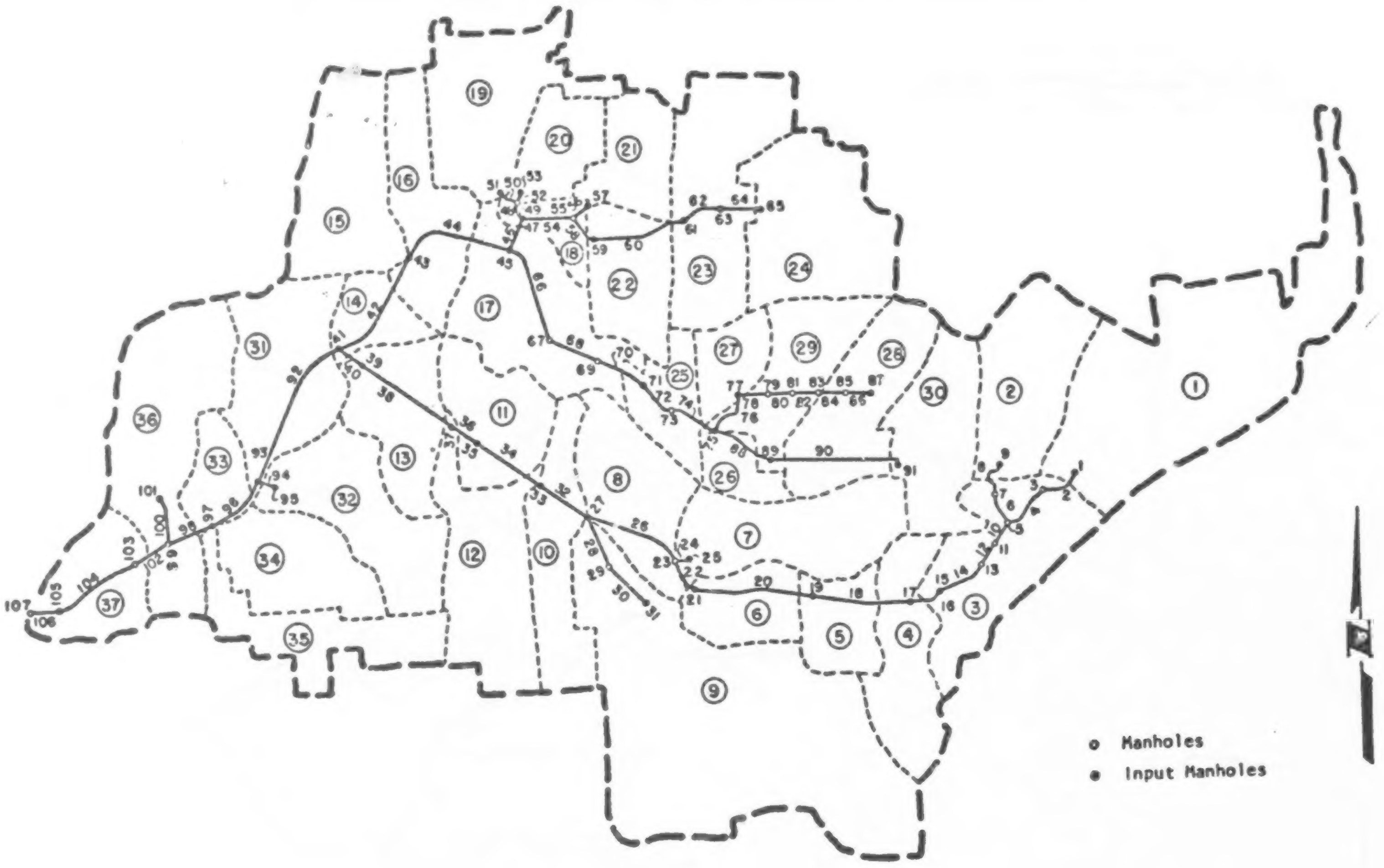

○ Manholes
● Input Manholes

Figure 13 . DIVISION OF BLOODY RUN SEWER BASIN INTO SUBAREAS AND NUMBERING OF THE SEWER SYSTEM'S ELEMENTS

Table 7. BLOODY RUN BASIN SEWER ELEMENTS

Element Number *	Location or Diameter	Slope	Contributing Subarea* or Length (feet)
1	input manhole		Drainage Subarea 1
2	54"	2.60%	394.0
3	manhole		
4	54"	2.30%	689.3
5	manhole		
6	48"	2.20%	380.4
7	manhole		
8	48"	2.60%	467.0
9	input manhole		Drainage Subarea 2
10	66"	1.40%	319.4
11	manhole		
12	72"	0.90%	345.3
13	manhole		
14	75"	1.00%	577.0
15	input manhole		Drainage Subarea 3
16	72"	1.40%	462.7
17	input manhole		Drainage Subarea 4
18	72"	1.80%	1428.5
19	input manhole		Drainage Subarea 5
20	78"	1.25%	1432.0
21	input manhole		Drainage Subarea 6
22	84"	1.10%	664.0
23	manhole		
24	60"	1.00%	149.7
25	input manhole		Drainage Subarea 7
26	90"	1.30%	1238.5
27	input manhole		Drainage Subarea 8
28	60"	1.20%	750.0
29	manhole		
30	60"	1.00%	700.0
31	input manhole		Drainage Subarea 9
32	10' x 8'	0.70%	700.0
33	input manhole		Drainage Subarea 10
34	10' x 8'	0.70%	900.0
35	input manhole		Drainage Subarea 11
36	10' x 8'	0.70%	438.3
37	input manhole		Drainage Subarea 12
38	11' x 9'	0.40%	1473.6
39	input manhole		Drainage Subarea 13
40	11' x 9'	0.40%	350.0
41	input manhole		Drainage Subarea 14
42	12' x 9'	0.30%	1673.1
43	input manhole		Drainage Subareas 15 & 16
44	12' x 9'	0.28%	1433.4
45	input manhole		Drainage Subarea 17
46	96"	0.60%	595.4

*referring to Figure 12.

Element Number	Location or Diameter	Slope	Contributing Subarea* or Length (feet)
47	input manhole		Drainage Subarea 18
48	54"	1.30%	291.2
49	manhole		
50	42"	2.40%	231.0
51	input manhole		Drainage Subarea 19
52	36"	3.40%	76.0
53	input manhole		Drainage Subarea 20
54	78"	0.90%	698.7
55	manhole		
56	42"	2.10%	209.3
57	input manhole		Drainage Subarea 21
58	78"	0.80%	442.8
59	input manhole		Drainage Subarea 22
60	66"	1.30%	1086.0
61	input manhole		Drainage Subarea 23
62	60"	1.50%	463.4
63	manhole		
64	54"	1.50%	475.5
65	input manhole		Drainage Subarea 24
66	78"	0.86%	1251.9
67	manhole		
68	72"	1.00%	930.4
69	manhole		
70	66"	1.90%	365.0
71	input manhole		Drainage Subarea 25
72	66"	2.30%	404.7
73	manhole		
74	66"	1.10%	800.2
75	input manhole		Drainage Subarea 26
76	48"	1.57%	481.0
77	input manhole		Drainage Subarea 27
78	48"	2.18%	354.4
79	manhole		
80	42"	5.70%	253.6
81	manhole		
82	36"	1.42%	389.4
83	manhole		
84	33"	1.67%	342.0
85	manhole	drop 24"	
86	27"	1.65%	325.9
87	input manhole		Drainage Subarea 28
88	48"	2.10%	858.0
89	input manhole		Drainage Subarea 29
90	42"	1.50%	1700.0
91	input manhole		Drainage Subarea 30
92	15' x 10'	0.28%	2190.0
93	input manhole		Drainage Subarea 31

Element Number	Location or Diameter	Slope	Contributing Subarea* or Length (feet)
94	48"	0.90%	170.0
95	input manhole		Drainage Subarea 32
96	15' x 10'	0.32%	775.0
97	input manhole		Drainage Subareas 33,34,35
98	15' x 10'	0.32%	495.0
99	manhole		
100	54"	0.50%	706.5
101	input manhole		Drainage Subarea 36
102	15' x 10'	0.32%	500.0
103	manhole		
104	15' x 10'	0.34%	1556.0
105	input manhole		Drainage Subarea 37
106	15' x 10'	0.34%	334.0
107	manhole and sewage outlet		

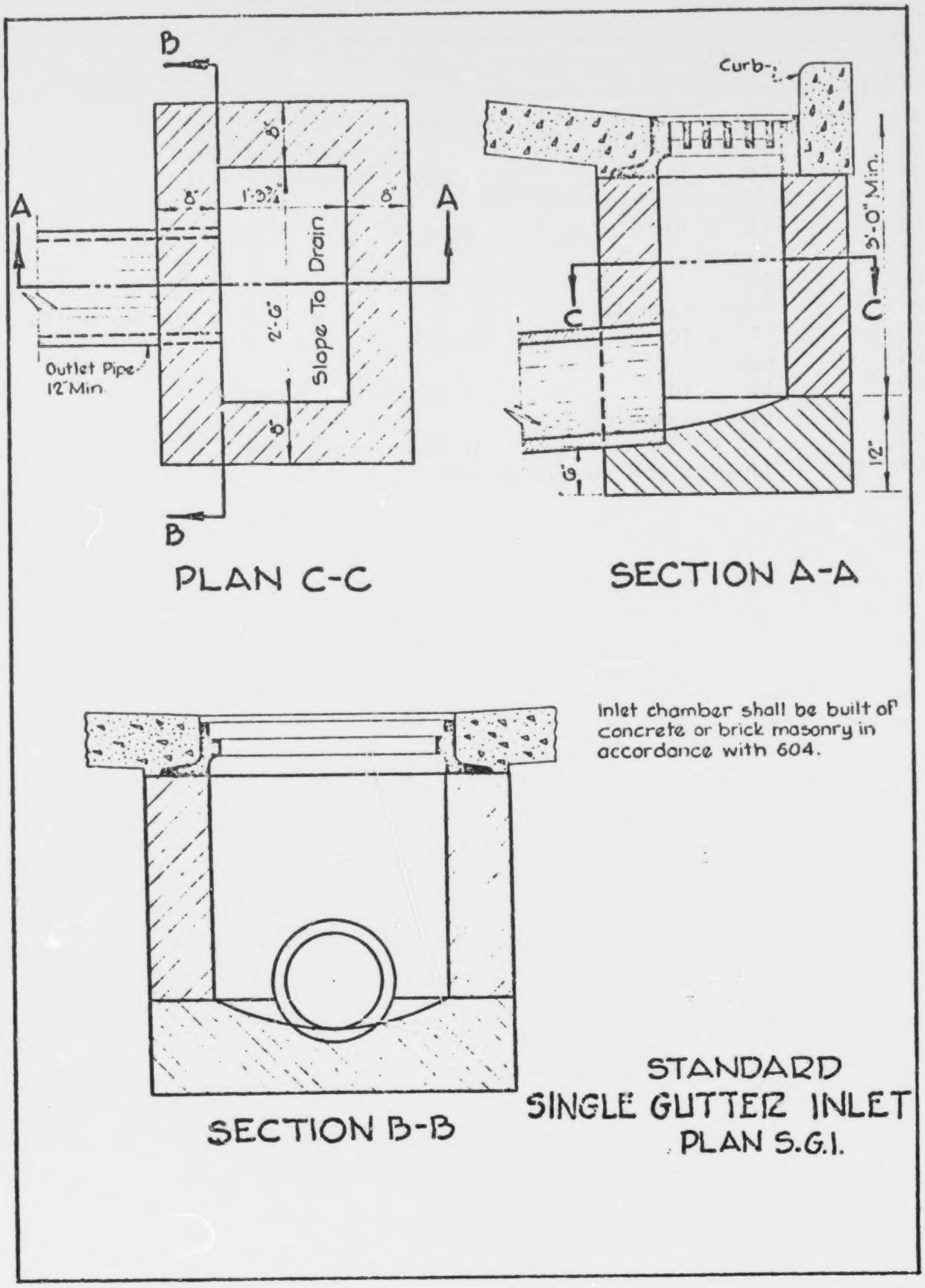

B
B
A
A
8"
1'-3¾"
8"
8"
2'-6"
Slope To Drain
8"
Outlet Pipe
12" Min.
PLAN C-C
Curb
3'-0" Min.
C
C
12"
6"
SECTION A-A
SECTION B-B
Inlet chamber shall be built of concrete or brick masonry in accordance with 604.
STANDARD
SINGLE GUTTER INLET
PLAN S.G.I.

Figure 14. STANDARD SINGLE GUTTER INLET

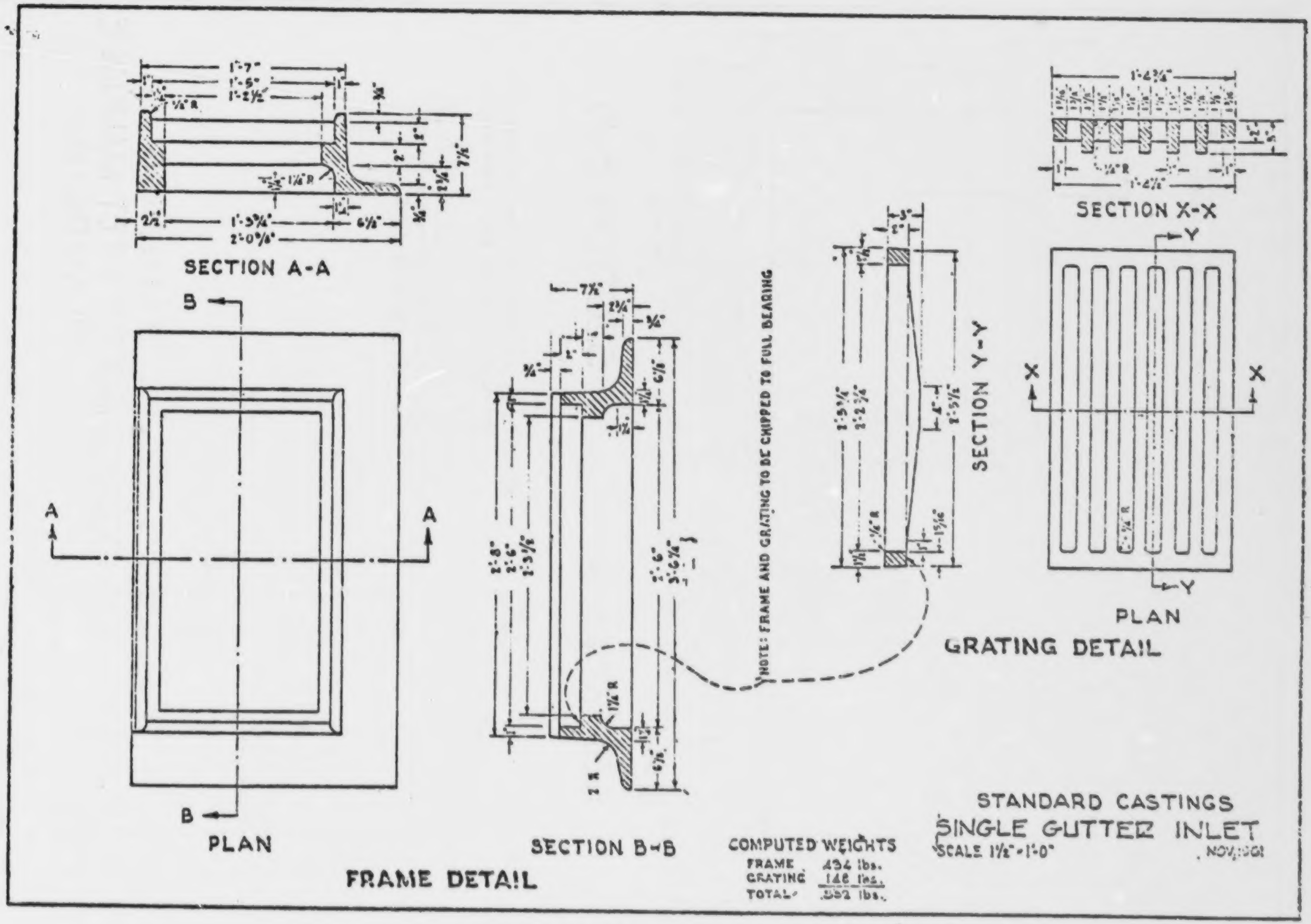

SECTION A-A
PLAN
FRAME DETAIL
SECTION B-B
NOTE: FRAME AND GRATING TO BE CHIPPED TO FULL BEARING
SECTION Y-Y
SECTION X-X
PLAN
GRATING DETAIL
COMPUTED WEIGHTS
FRAME 434 lbs.
GRATING 148 lbs.
TOTAL 582 lbs.
STANDARD CASTINGS
SINGLE GUTTER INLET
SCALE 1½"=1'-0"
NOV. 1961

Figure 15. STANDARD CASTINGS SINGLE GUTTER INLET

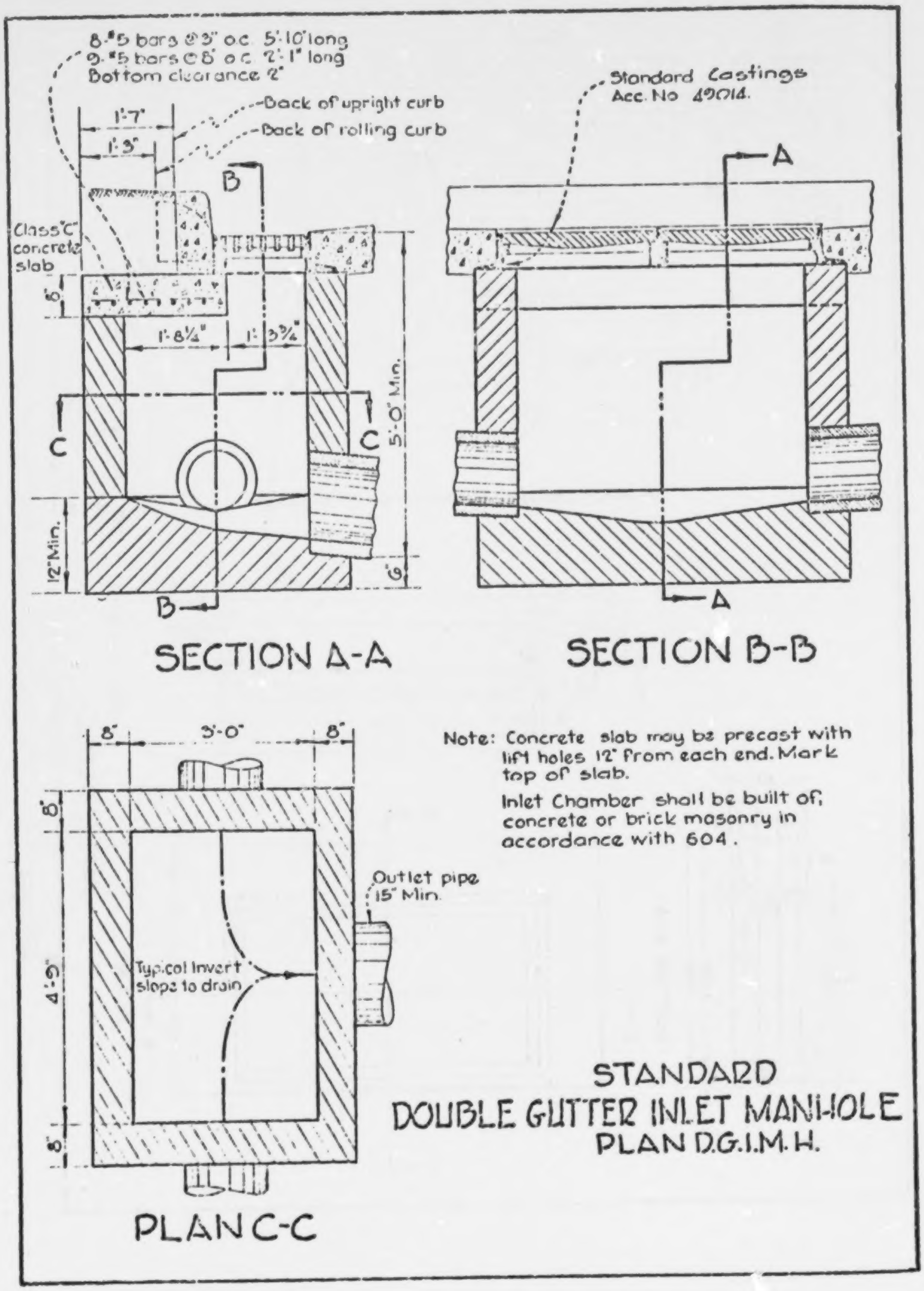

8-#5 bars @ 3" o.c. 5'-10" long
9-#5 bars @ 8" o.c. 2'-1" long
Bottom clearance 2"
Back of upright curb
Back of rolling curb
1'-7"
1'-3"
B
Class "C" concrete slab
6"
1'-8¼"
1'-3¾"
5'-0" Min.
C
C
12" Min.
6"
B
SECTION A-A
Standard Castings Acc. No. 49014.
A
A
SECTION B-B
8"
3'-0"
8"
8"
4'-5"
8"
Outlet pipe 15" Min.
Typical Invert slope to drain
PLAN C-C
Note: Concrete slab may be precast with lift holes 12" from each end. Mark top of slab.
Inlet Chamber shall be built of concrete or brick masonry in accordance with 604.
STANDARD
DOUBLE GUTTER INLET MANHOLE
PLAN D.G.I.M.H.

Figure 16 . STANDARD DOUBLE GUTTER INLET MANHOLE

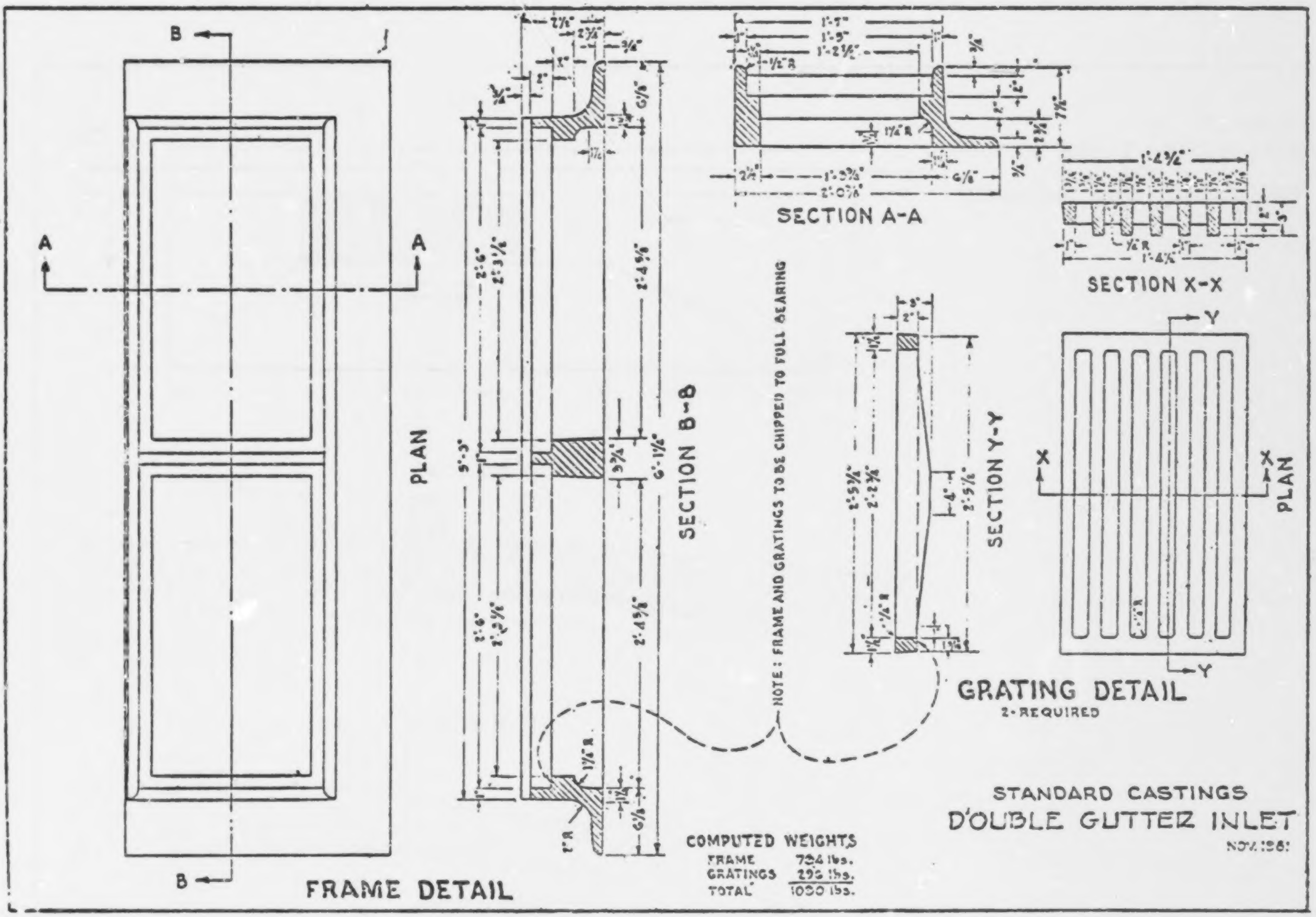

FRAME DETAIL
PLAN
SECTION B-B
SECTION A-A
SECTION X-X
SECTION Y-Y
GRATING DETAIL
2-REQUIRED
NOTE : FRAME AND GRATINGS TO BE CHIPPED TO FULL BEARING
COMPUTED WEIGHTS
FRAME 734 lbs.
GRATINGS 296 lbs.
TOTAL 1030 lbs.
STANDARD CASTINGS
DOUBLE GUTTER INLET
NOV. 1961
A
B
X
Y
5'-3"
6'-1½"

Figure 17 . STANDARD CASTINGS DOUBLE GUTTER INLET

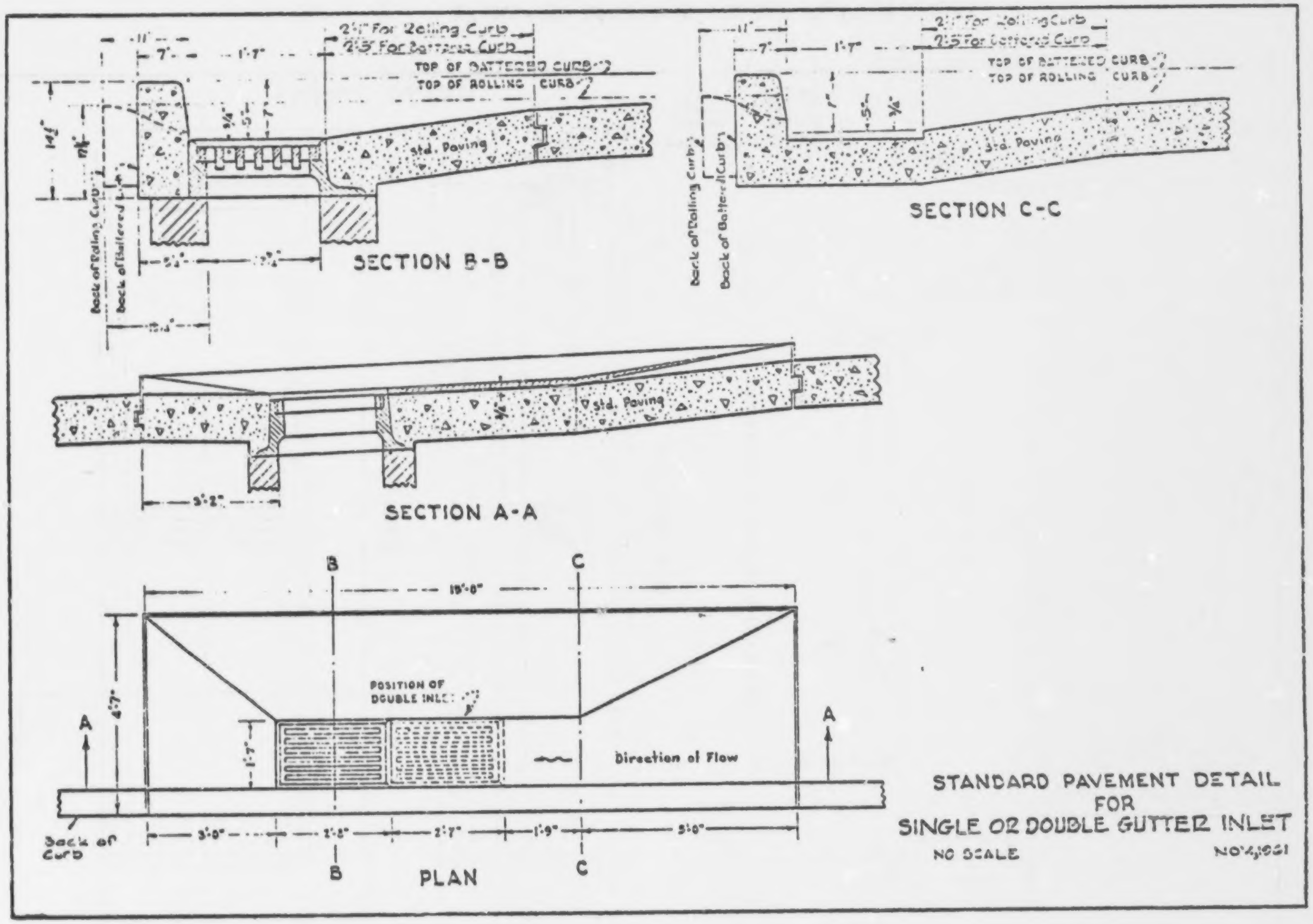
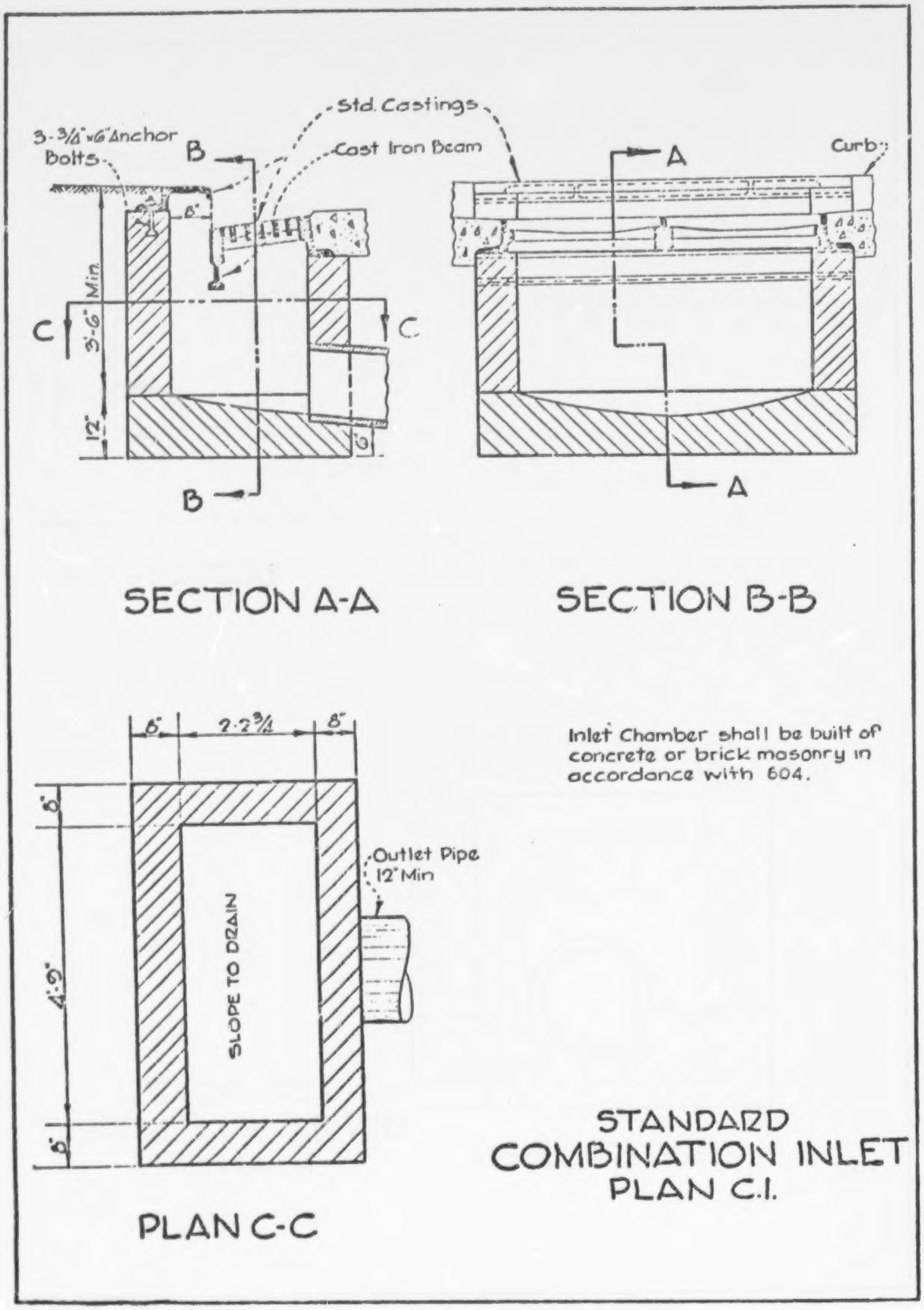

2'-1" For Rolling Curb
2'-3" For Battered Curb
TOP OF BATTERED CURB
TOP OF ROLLING CURB
11"
7"
1'-7"
Std. Paving
Back of Rolling Curb
Back of Battered Curb
SECTION B-B
SECTION C-C
SECTION A-A
5'-2"
B
C
A
15'-0"
4'-7"
1'-7"
POSITION OF DOUBLE INLET
Direction of Flow
Back of Curb
3'-0"
2'-8"
2'-7"
1'-9"
5'-0"
PLAN
STANDARD PAVEMENT DETAIL
FOR
SINGLE OR DOUBLE GUTTER INLET
NO SCALE
NOV. 1951

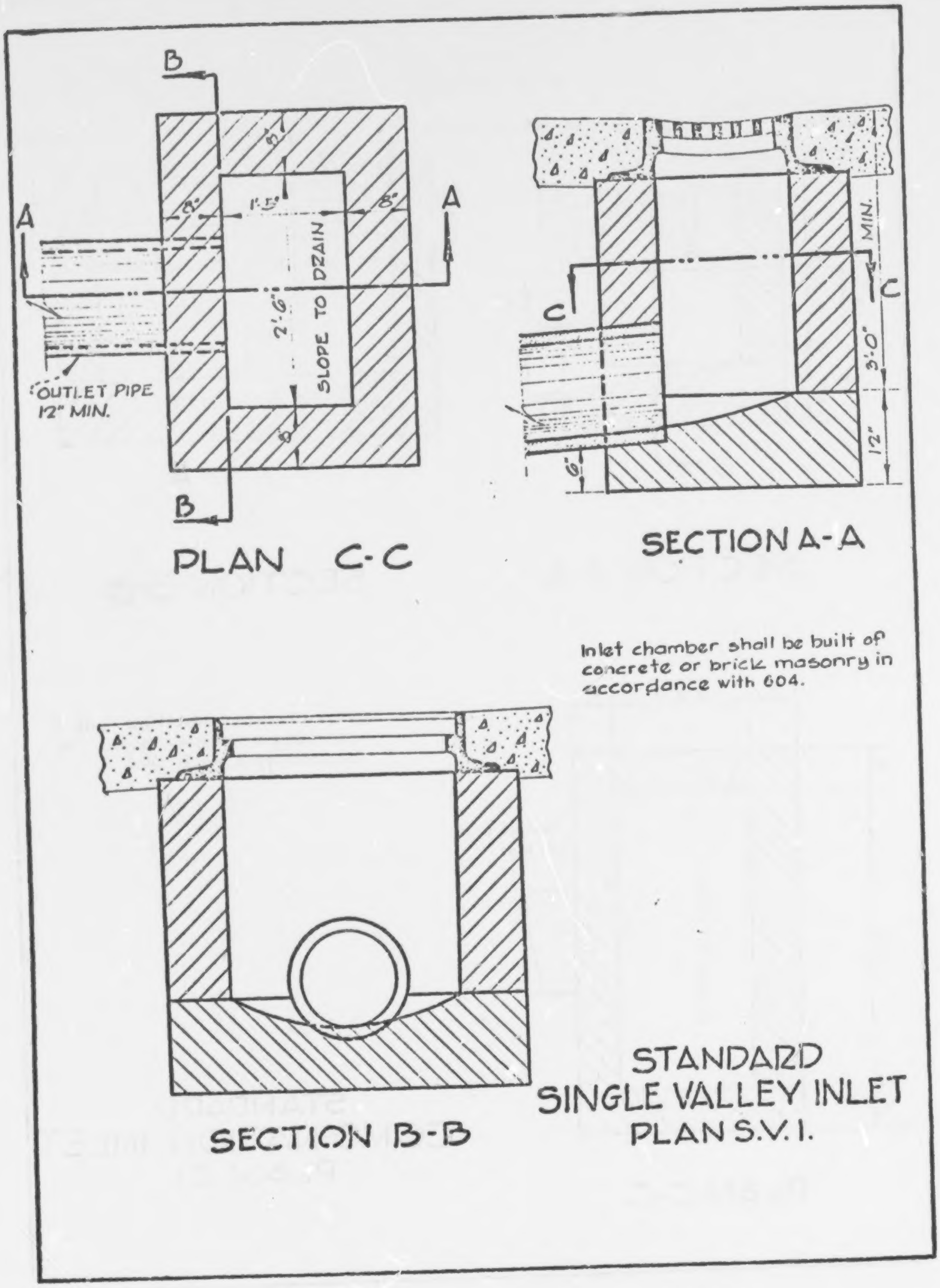
Figure 18. STANDARD PAVEMENT DETAIL FOR SINGLE OR DOUBLE GUTTER INLET

NOT REPRODUCIBLE


3-3/4"×6" Anchor Bolts
Std. Castings
Cast Iron Beam
Curb
B
B
A
A
C
C
8"
3'-6" Min.
12"
6"
SECTION A-A
SECTION B-B
6"
2-2 3/4
8"
8"
4'-5"
6"
SLOPE TO DRAIN
Outlet Pipe 12" Min
PLAN C-C
STANDARD
COMBINATION INLET
PLAN C.I.

Inlet Chamber shall be built of concrete or brick masonry in accordance with 604.

Figure 19 . STANDARD COMBINATION INLET



B
B
A
A
8"
1'-3"
8"
8"
2'-6"
SLOPE TO DRAIN
8"
OUTLET PIPE 12" MIN.
PLAN C-C
MIN.
C
C
3'-0"
12"
6"
SECTION A-A
SECTION B-B
STANDARD SINGLE VALLEY INLET PLAN S.V.I.

Inlet chamber shall be built of concrete or brick masonry in accordance with 604.

Figure 20 . STANDARD SINGLE VALLEY INLET

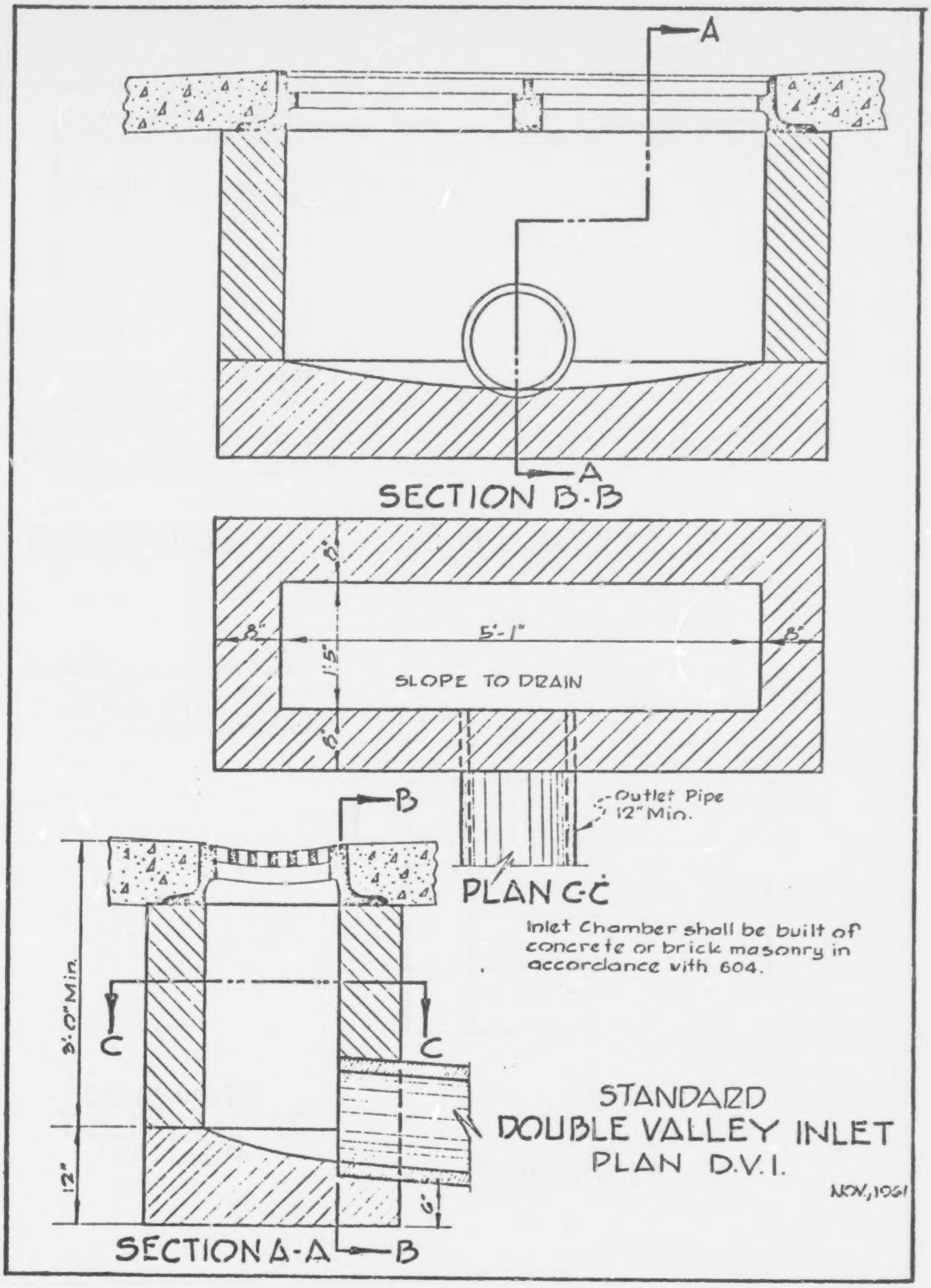

A
A
SECTION B-B
6"
8"
5'-1"
8"
1'5"
SLOPE TO DRAIN
6"
B
Outlet Pipe 12" Min.
PLAN C-C
Inlet Chamber shall be built of concrete or brick masonry in accordance with 604.
3'-0" Min.
C
C
12"
6"
STANDARD
DOUBLE VALLEY INLET
PLAN D.V.I.
NOV, 1961
SECTION A-A
B

Figure 21. STANDARD DOUBLE VALLEY INLET

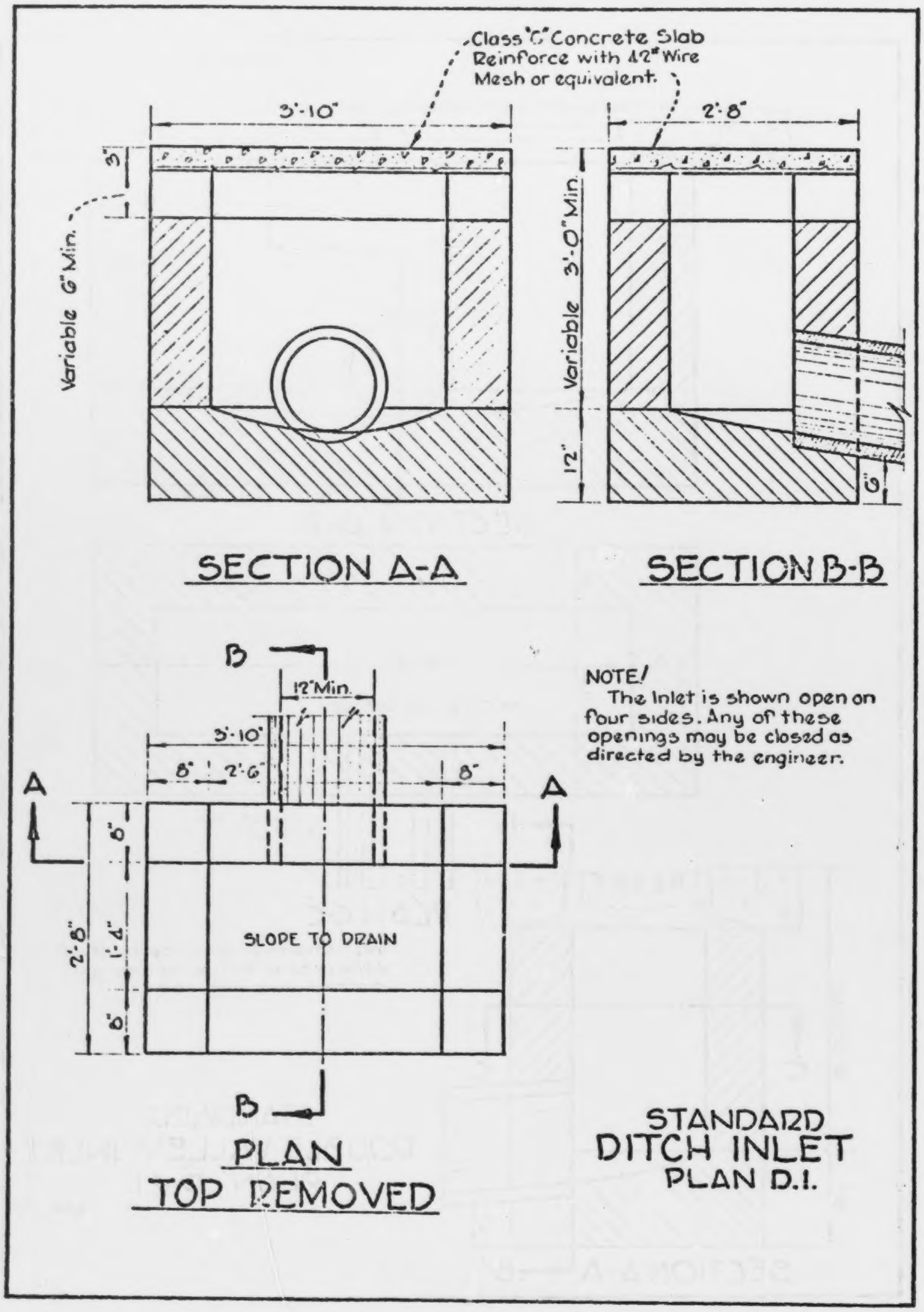

Class "C" Concrete Slab
Reinforce with 12# Wire
Mesh or equivalent.
3'-10"
2'-8"
3"
Variable 6" Min.
Variable 3'-0" Min.
12"
6"
SECTION A-A
SECTION B-B
B
12" Min.
3'-10"
8"
2'-6"
8"
A
A
8"
2'-8"
1'-4"
8"
SLOPE TO DRAIN
B
PLAN
TOP REMOVED
NOTE!
The Inlet is shown open on
four sides. Any of these
openings may be closed as
directed by the engineer.
STANDARD
DITCH INLET
PLAN D.I.

Figure 22. STANDARD DITCH INLET

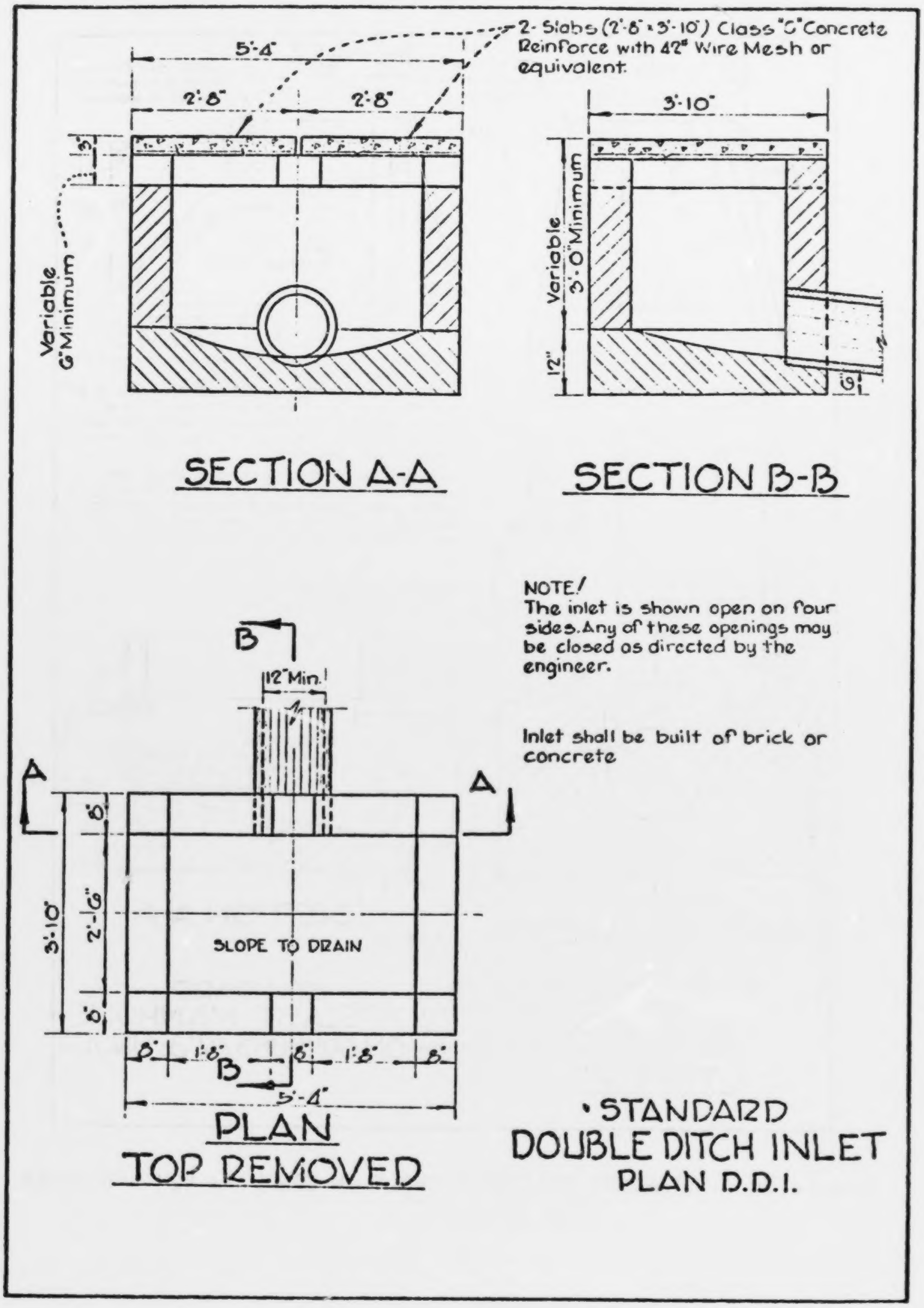
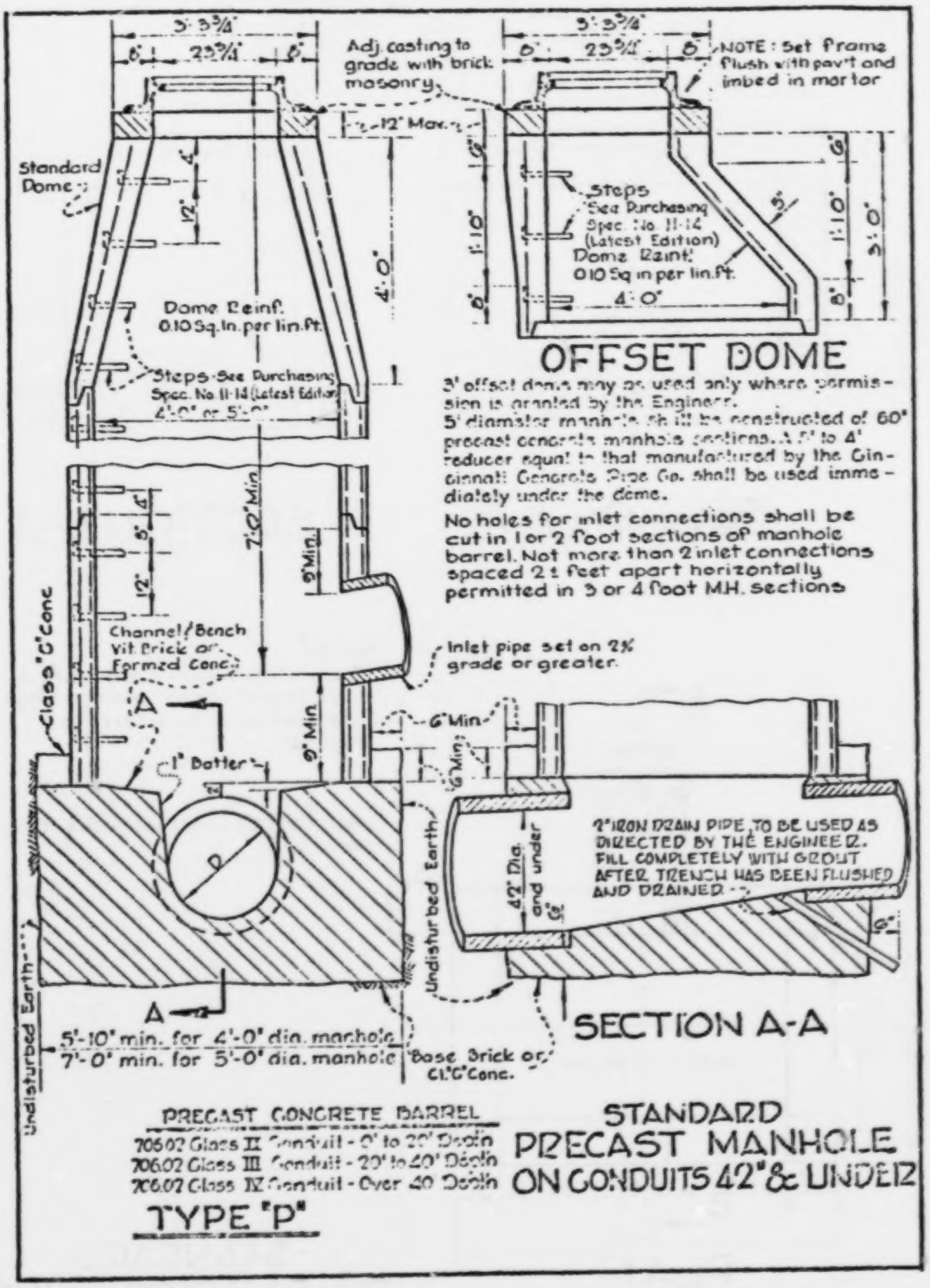

2- Slabs (2'-8" x 3'-10") Class "C" Concrete Reinforce with 42# Wire Mesh or equivalent.
5'-4"
2'-8"
2'-8"
3"
Variable 6" Minimum
3'-10"
Variable 3'-0" Minimum
12"
6"
SECTION A-A
SECTION B-B
NOTE!
The inlet is shown open on four sides. Any of these openings may be closed as directed by the engineer.
Inlet shall be built of brick or concrete
B
12" Min.
A
A
8"
2'-6"
8"
3'-10"
SLOPE TO DRAIN
8"
1'-8"
B
8"
1'-8"
8"
5'-4"
PLAN
TOP REMOVED
• STANDARD
DOUBLE DITCH INLET
PLAN D.D.I.

Figure 23 . STANDARD DOUBLE DITCH INLET

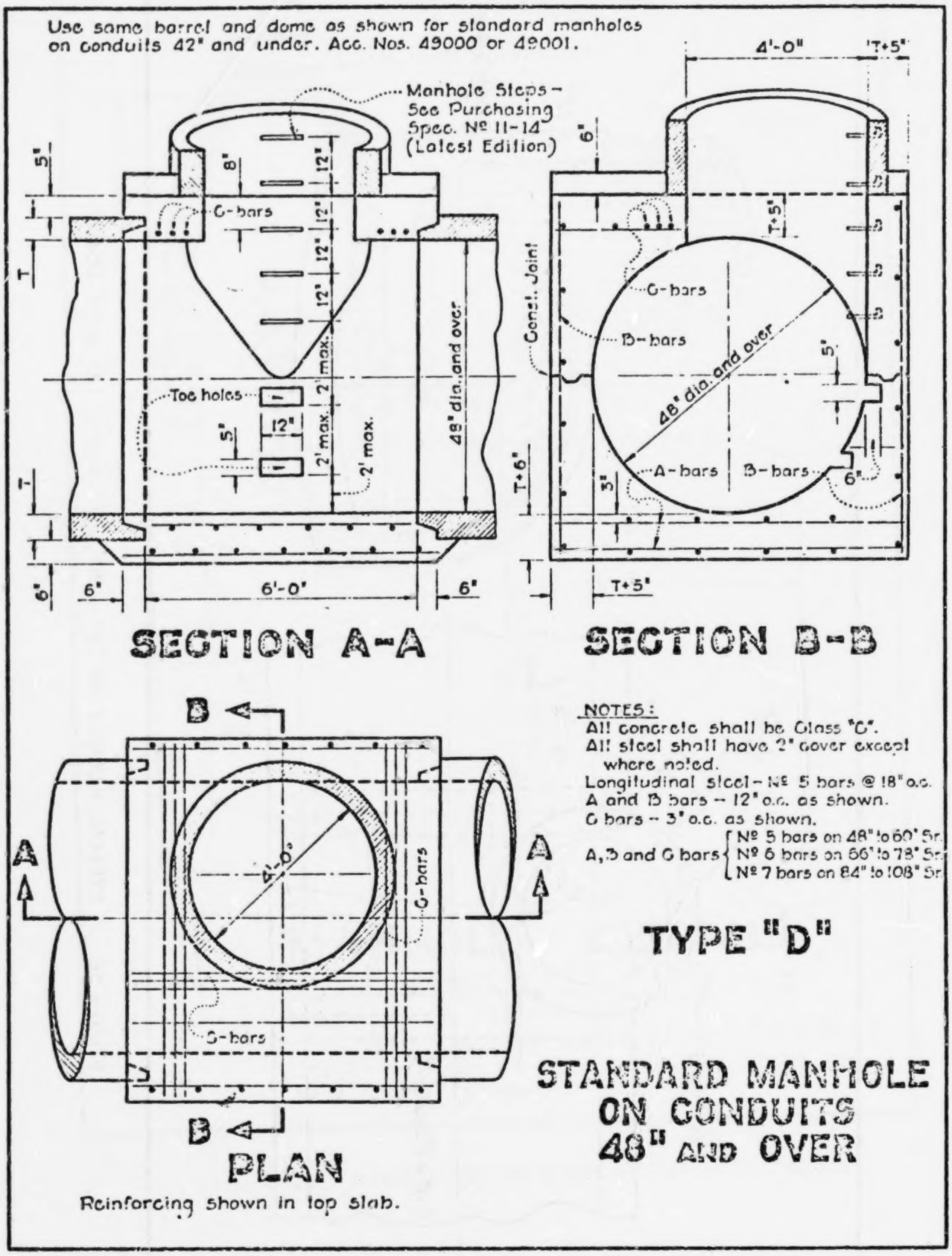

3'-3¾"
8"
23¾"
8"
Adj. casting to grade with brick masonry
12" Max.
Standard Dome
4'
12"
4'-0"
Dome Reinf. 0.10 Sq. In. per lin. ft.
Steps - See Purchasing Spec. No. 11-14 (Latest Edition)
4'-0" or 5'-0"
NOTE: Set frame flush with pav't and imbed in mortar
Steps See Purchasing Spec. No. 11-14 (Latest Edition)
Dome Reinf. 0.10 Sq in per lin. ft.
4'-0"
3'
6"
1'-10"
8"
3'-0"
OFFSET DOME
7'-0" Min
9" Min.
4'
3'
12"
Class "C" Conc
Channel/Bench Vit. Brick or Formed Conc.
Inlet pipe set on 2% grade or greater.
A
9" Min
1" Batter
6" Min
6" Min
D
Undisturbed Earth
42" Dia. and under
2" IRON DRAIN PIPE TO BE USED AS DIRECTED BY THE ENGINEER. FILL COMPLETELY WITH GROUT AFTER TRENCH HAS BEEN FLUSHED AND DRAINED
6"
A
5'-10" min. for 4'-0" dia. manhole
7'-0" min. for 5'-0" dia. manhole
Base Brick or Cl. "C" Conc.
Undisturbed Earth
SECTION A-A
PRECAST CONCRETE BARREL
706.02 Class II Conduit - 0' to 20' Depth
706.02 Class III Conduit - 20' to 40' Depth
706.02 Class IV Conduit - Over 40 Depth
TYPE "P"
STANDARD PRECAST MANHOLE ON CONDUITS 42" & UNDER

3' offset dome may be used only where permission is granted by the Engineer.
5' diameter manhole shall be constructed of 60" precast concrete manhole sections. A 5' to 4' reducer equal to that manufactured by the Cincinnati Concrete Pipe Co. shall be used immediately under the dome.

No holes for inlet connections shall be cut in 1 or 2 foot sections of manhole barrel. Not more than 2 inlet connections spaced 2± feet apart horizontally permitted in 3 or 4 foot M.H. sections

Figure 24 . STANDARD PRECAST MANHOLE ON CONDUITS 42" AND UNDER

NOT REPRODUCIBLE


Use same barrel and dome as shown for standard manholes on conduits 42" and under. Acc. Nos. 49000 or 49001.

Manhole Steps – See Purchasing Spec. Nº 11-14 (Latest Edition)
4'-0"
T+5"
5'
8"
12"
12"
12"
12"
C-bars
T
Toe holes
2' max.
2' max.
2' max.
12"
5"
48" dia. and over
6"
6'-0"
6"
6"
T+5"
Const. Joint
C-bars
B-bars
48" dia. and over
5'
A-bars
B-bars
B-bars
6"
3"
T+6"
T+5"
SECTION A-A
SECTION B-B
B
B
A
A
4'-0"
C-bars
C-bars
PLAN
TYPE "D"

NOTES:
All concrete shall be Class "C".
All steel shall have 2" cover except where noted.
Longitudinal steel – Nº 5 bars @ 18" o.c.
A and B bars – 12" o.c. as shown.
C bars – 3" o.c. as shown.
A, B and C bars { Nº 5 bars on 48" to 60" Sr. / Nº 6 bars on 66" to 78" Sr. / Nº 7 bars on 84" to 108" Sr.

Reinforcing shown in top slab.

STANDARD MANHOLE ON CONDUITS 48" AND OVER

Figure 25. STANDARD MANHOLE ON CONDUITS 48" AND OVER

NOT REPRODUCIBLE

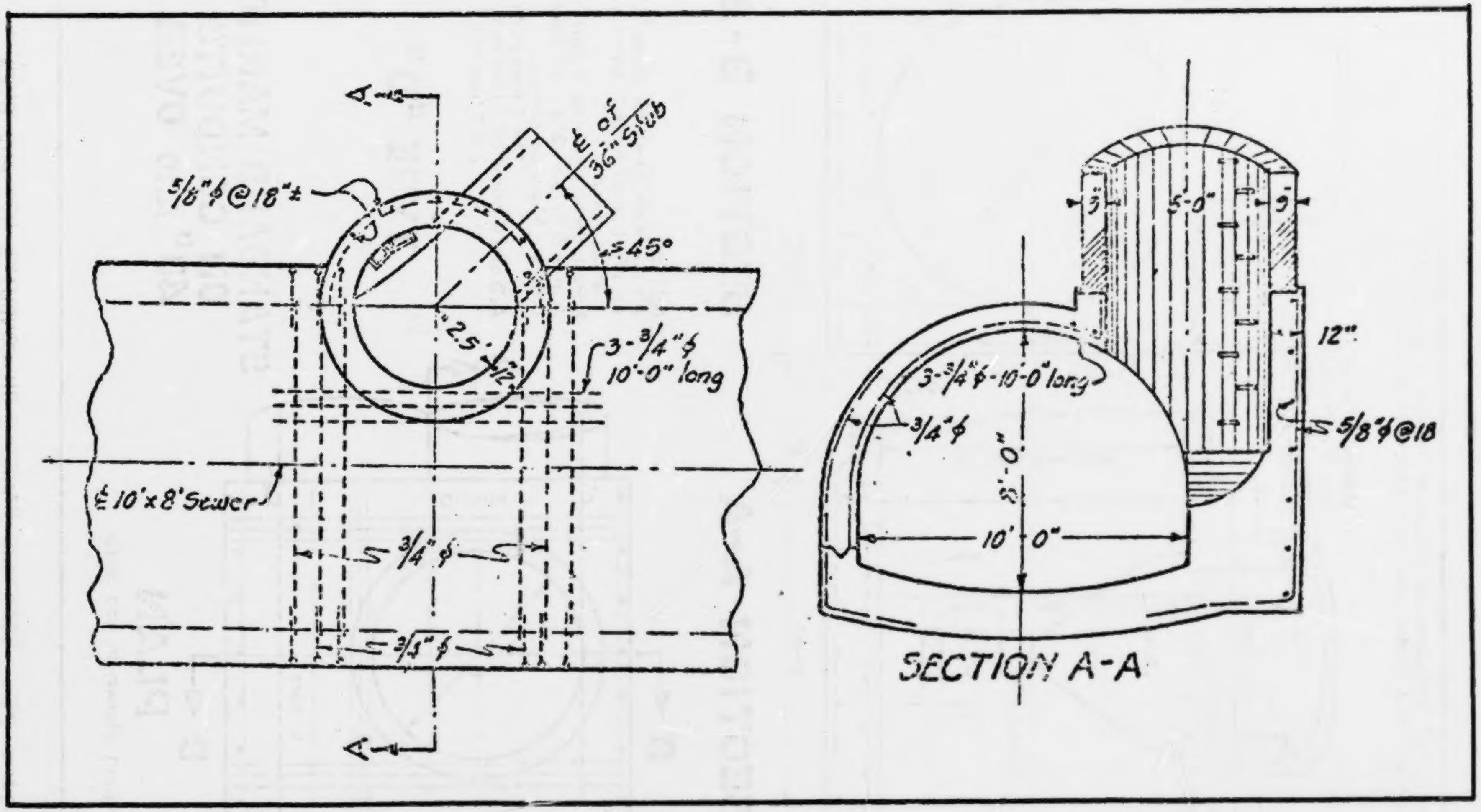
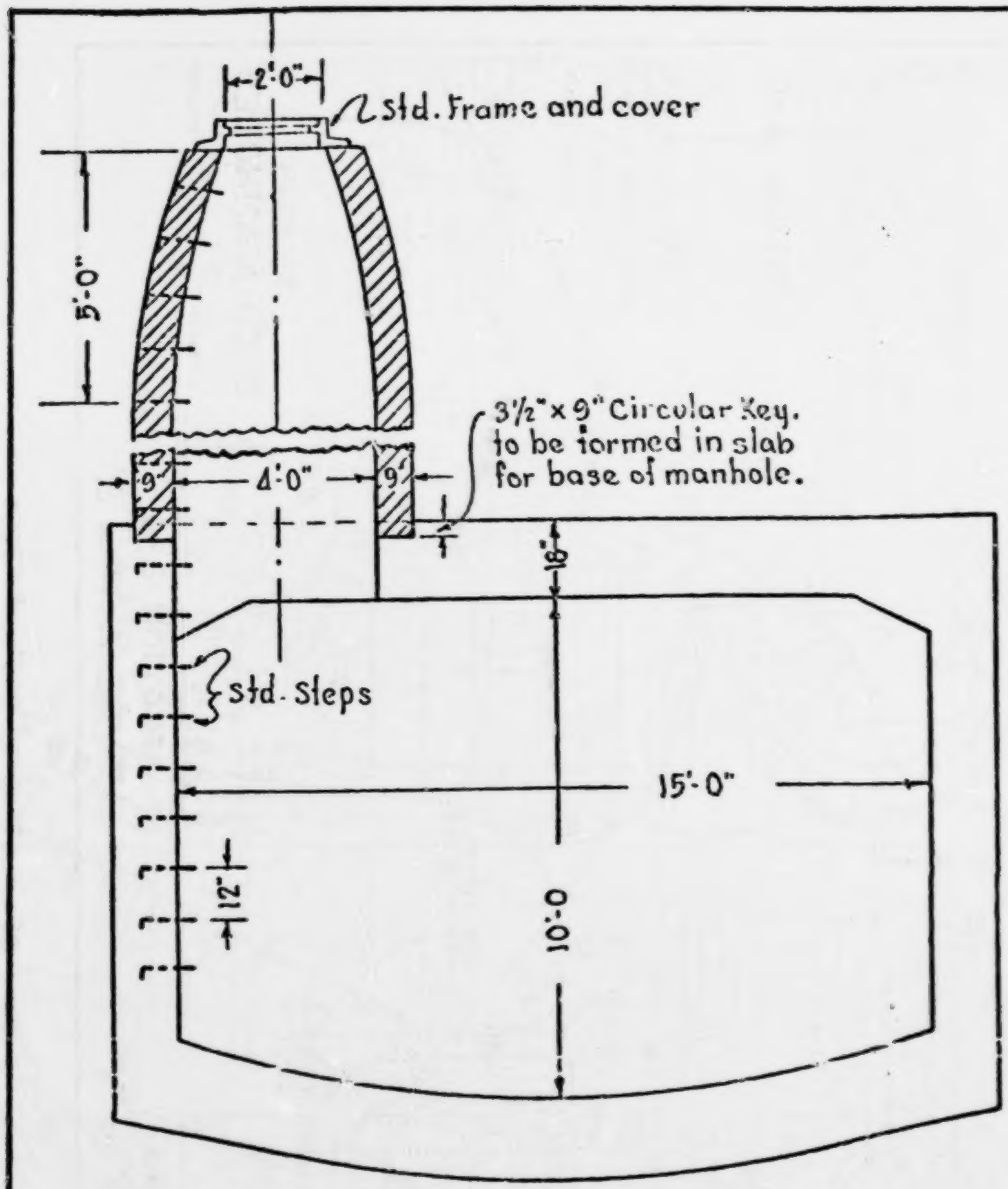

A
5/8"φ@18"±
℄ of 36" Stub
45°
2.5'
1/2
3-3/4"φ 10'-0" long
℄ 10'x8' Sewer
3/4"φ
3/4"φ
A
3
5'-0"
9
12"
3-3/4"φ-10'-0" long
3/4"φ
8'-0"
5/8"φ@18
10'-0"
SECTION A-A

Figure 26 . TYPICAL MANHOLE ON 10' x 8', 11' x 9', 12' x 9' SEWERS


2'-0"
Std. Frame and cover
5'-0"
3½" x 9" Circular Key.
to be formed in slab
for base of manhole.
9"
4'-0"
9"
1"
16"
Std. Steps
15'-0"
12"
10'-0

MANHOLE DETAIL

SCALE ¼" = 1'-0"

NOTE:-

The customary deduction of 4.0 ft. of sewer will not be made for manholes on 15'-0" x 10'-0" sewer.

Figure 27. MANHOLE ON RECTANGULAR 10' x 15' SEWER

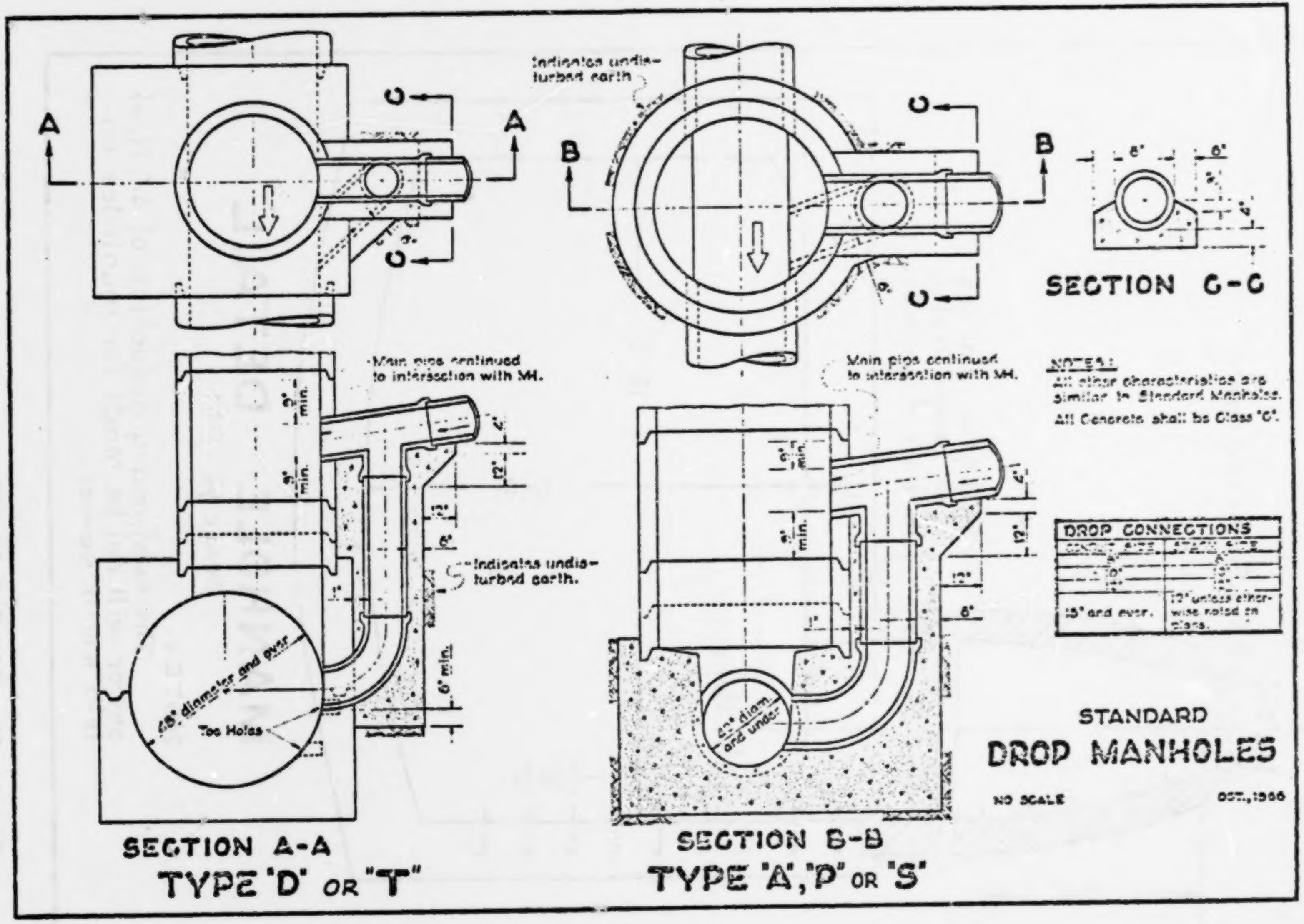

A
A
C
C
B
B
C
C
Indicates undis-
turbed earth
SECTION C-C
6"
6"
Main pipe continued
to intersection with MH.
Main pipe continued
to intersection with MH.
Indicates undis-
turbed earth.
48" diameter and over
Toe Holes
42" diam. and under
6" min.
12"
4"
1"
SECTION A-A
TYPE 'D' OR "T"
SECTION B-B
TYPE 'A','P' OR 'S'
NOTES:
All other characteristics are
similar to Standard Manholes.
All Concrete shall be Class "C".
DROP CONNECTIONS
15" and over.
12" unless other-
wise noted on
plans.
STANDARD
DROP MANHOLES
NO SCALE
OCT.,1966

Figure 28 . STANDARD DROP MANHOLES

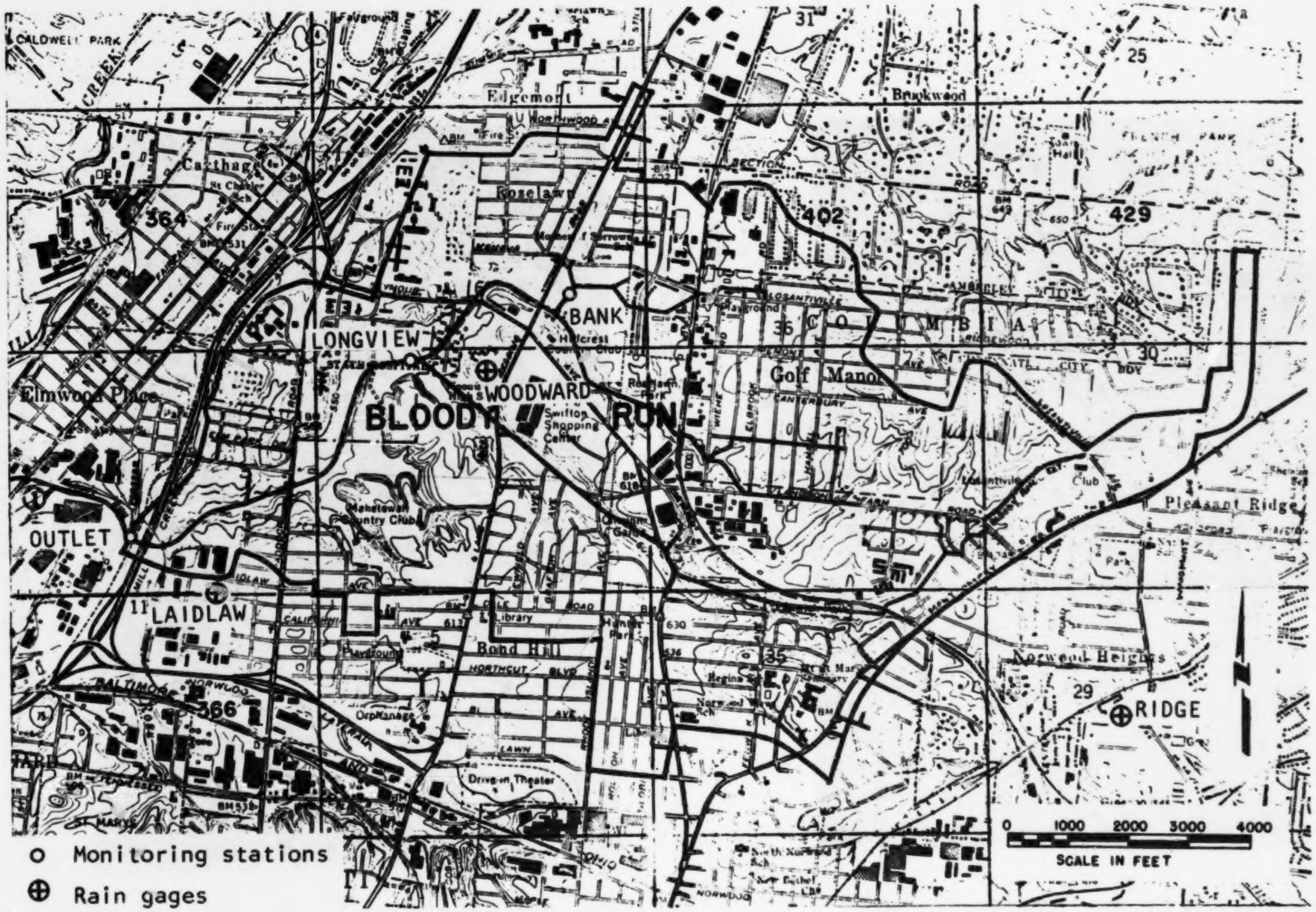

BLOODY RUN
WOODWARD
LONGVIEW
BANK
OUTLET
LAIDLAW
RIDGE
Edgemont
Roselawn
Carthage
Elmwood Place
Golf Manor
Bond Hill
Pleasant Ridge
Norwood Heights
COLUMBIA
Swifton Shopping Center
CALDWELL PARK
364
366
402
429
0 1000 2000 3000 4000
SCALE IN FEET
o Monitoring stations
⊕ Rain gages

Figure 29. BLOODY RUN SEWER WATERSHED MONITORING STATIONS AND RAIN GAGES

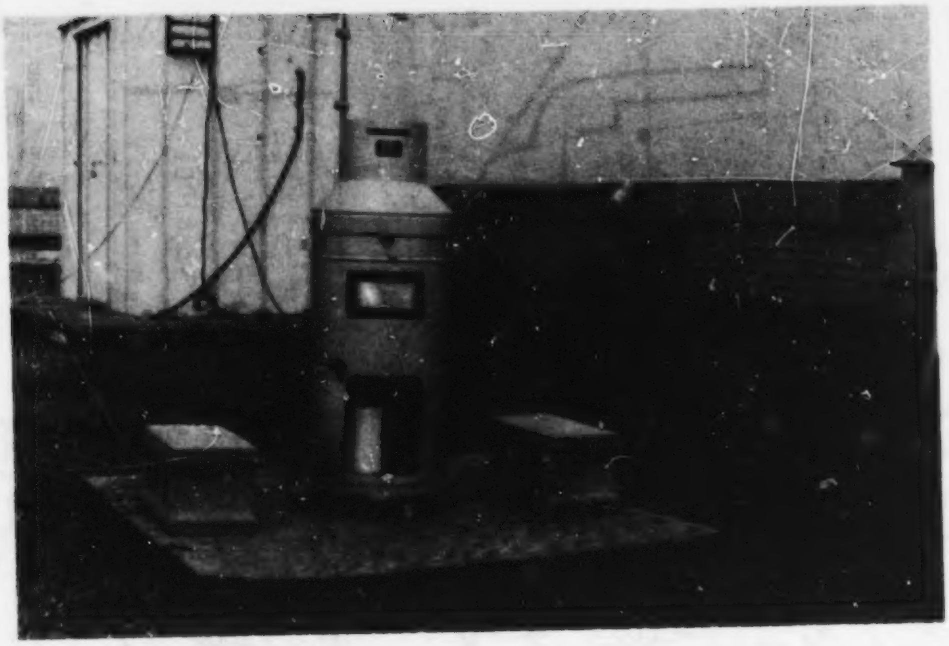


Figure 30.

TYPICAL RAINGAGE UNIT, ON LAIDLAW AVENUE NEAR THE OUTLET OF BLOODY RUN BASIN

Figure 31.

MONITORING STATION BANK NEXT TO SWIFTON SHOPPING CENTER

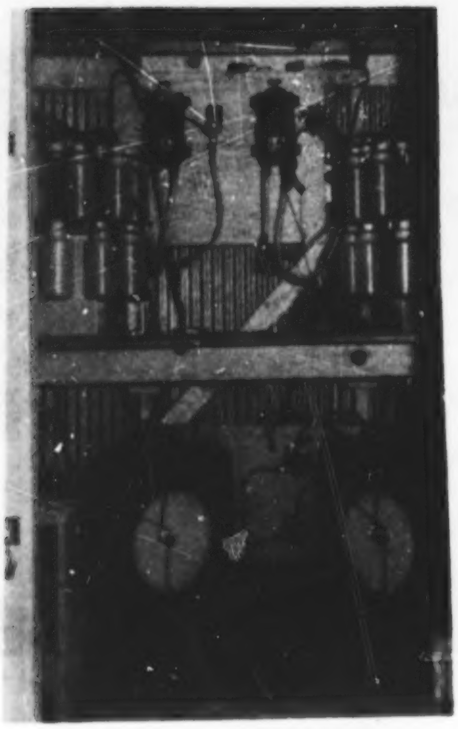


Figure 32.

BANK MONITORING STATION
SAMPLING AND FLOW MEASURING APPARATUS
CONNECTED WITH A 54" AND A 78" SEWER

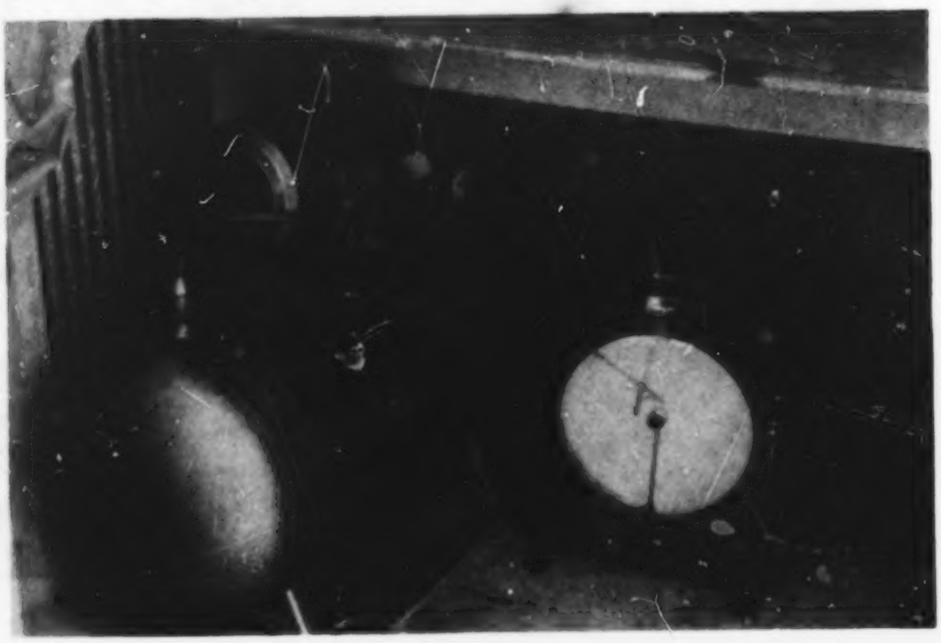


Figure 33.

BANK MONITORING STATION
AIR-COMPRESSOR AND TAYLOR FLOW RECORDER

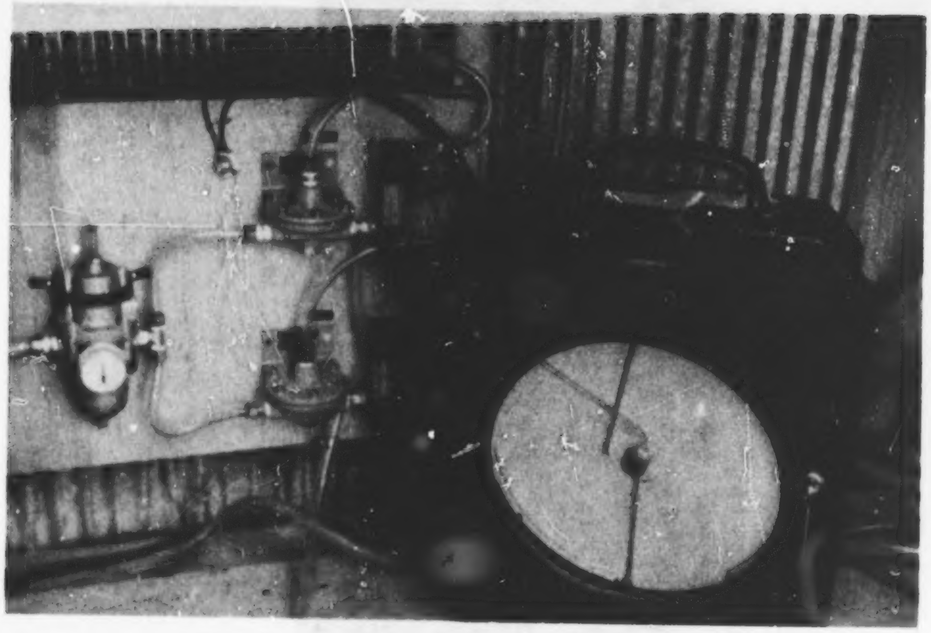


Figure 34.

BANK MONITORING STATION
MANOMETERS AND TAYLOR FLOW RECORDER

Figure 35.

BANK MONITORING STATION. JUNCTION OF TWO SEWER PIPES,
54" AND 78" IN DIAMETER

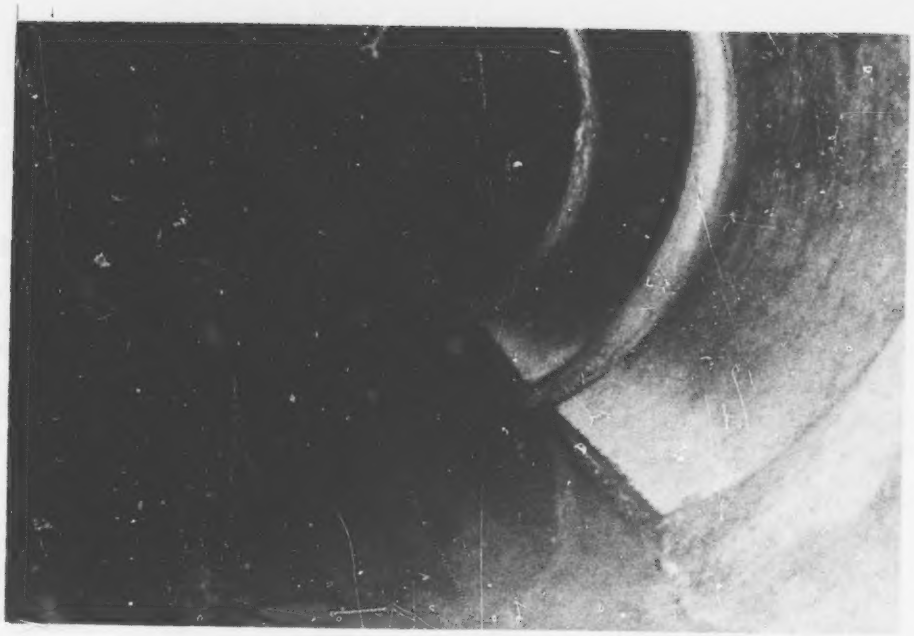


Figure 36 .
SEWER, 78" IN DIAMETER, DURING DRY WEATHER FLOW
(BANK MONITORING STATION)

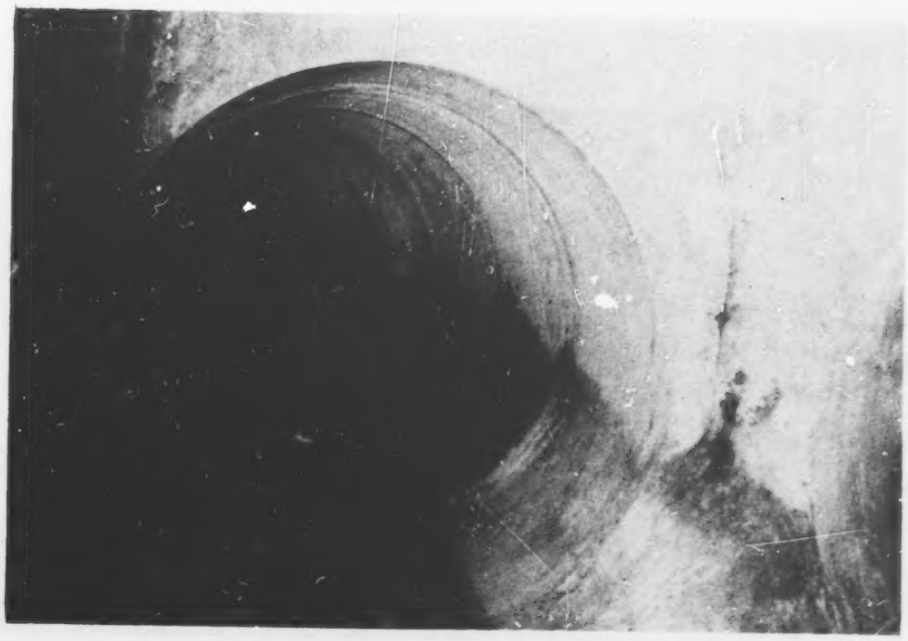


Figure 37 .
SEWER, 54" IN DIAMETER, DURING DRY WEATHER FLOW
(BANK MONITORING STATION)

Figure 38 .

GENERAL VIEW OF THE LONGVIEW MONITORING STATION AREA

Figure 39 .

LONGVIEW MONITORING STATION

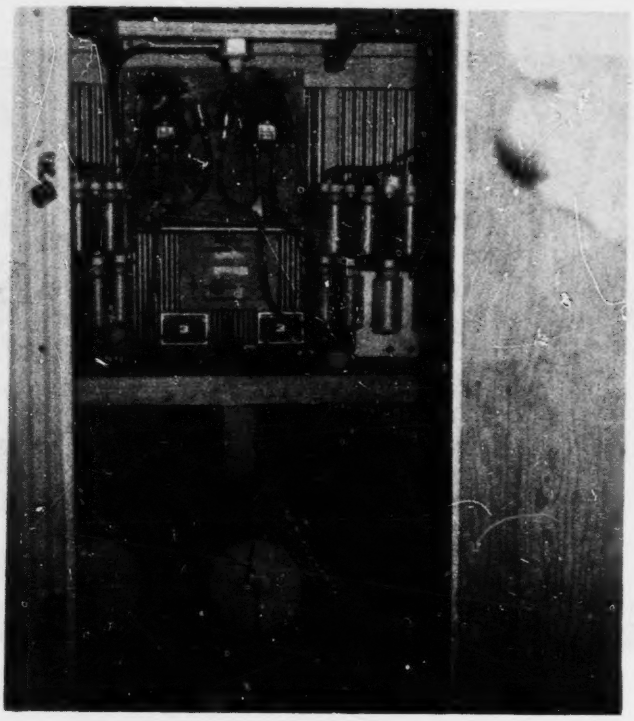


Figure 40 .

LONGVIEW MONITORING STATION

SAMPLING AND FLOW MEASURING APPARATUS

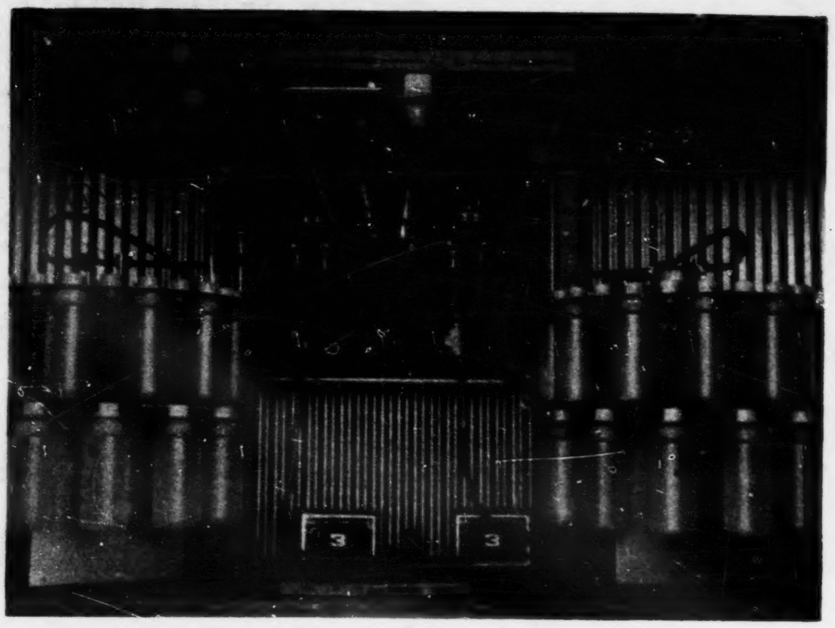


Figure 41 .

LONGVIEW MONITORING STATION
THE TWO SAMPLING INSTALLATIONS CONNECTED
WITH 11' x 9' AND 12' x 9' SEWERS

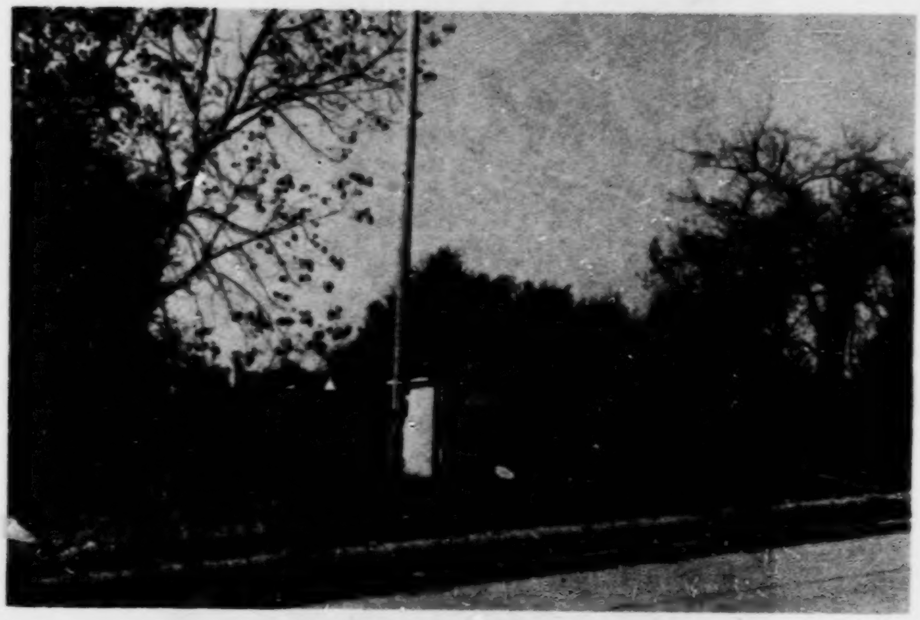


Figure 42 .

OUTLET MONITORING STATION ON PROSSER STREET

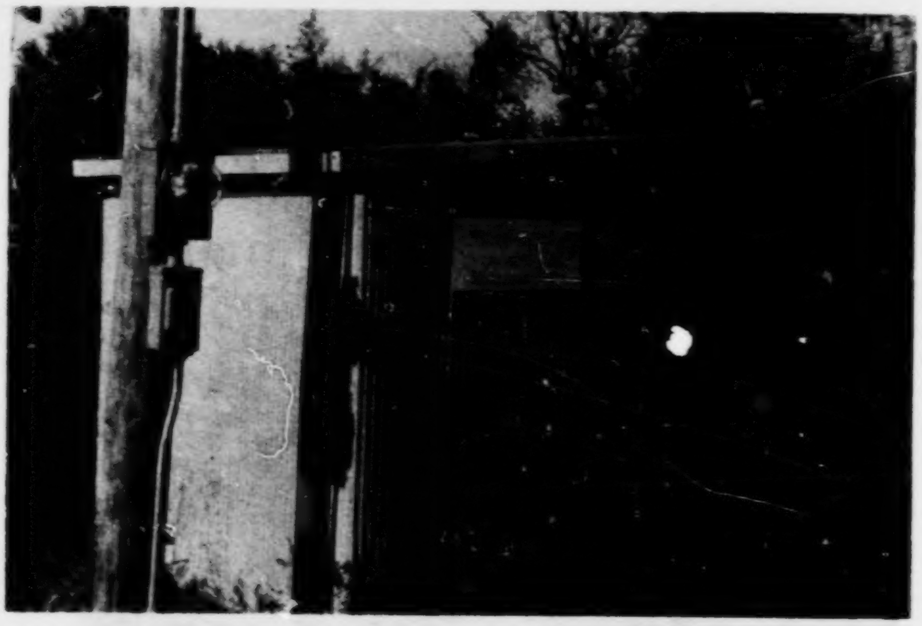


Figure 43 .

OUTLET MONITORING STATION AND POWER POLE

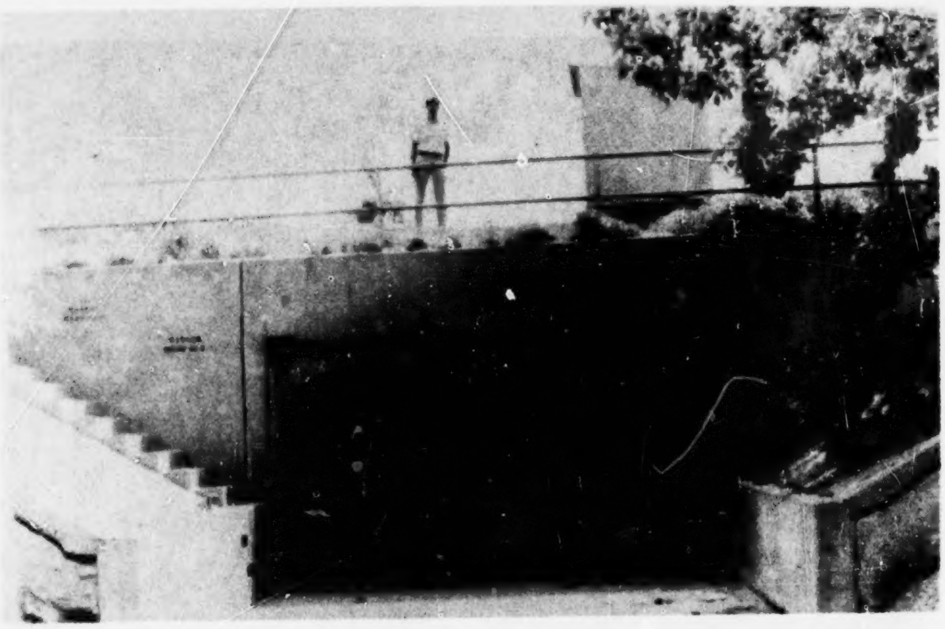


Figure 44 .

STORM WATER OUTFALL FROM THE BLOODY RUN BASIN

AT THE OUTLET MONITORING STATION

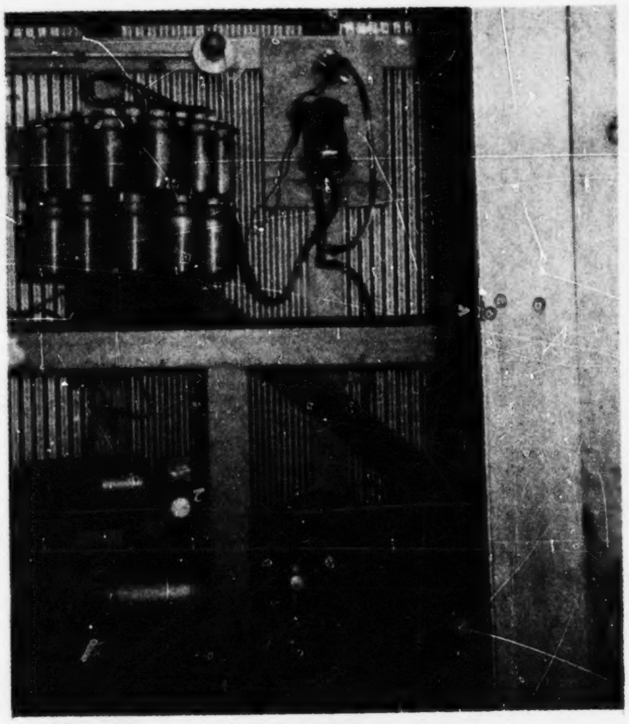


Figure 45 .

OUTLET MONITORING STATION

FLOW MEASURING AND SAMPLING APPARATUS

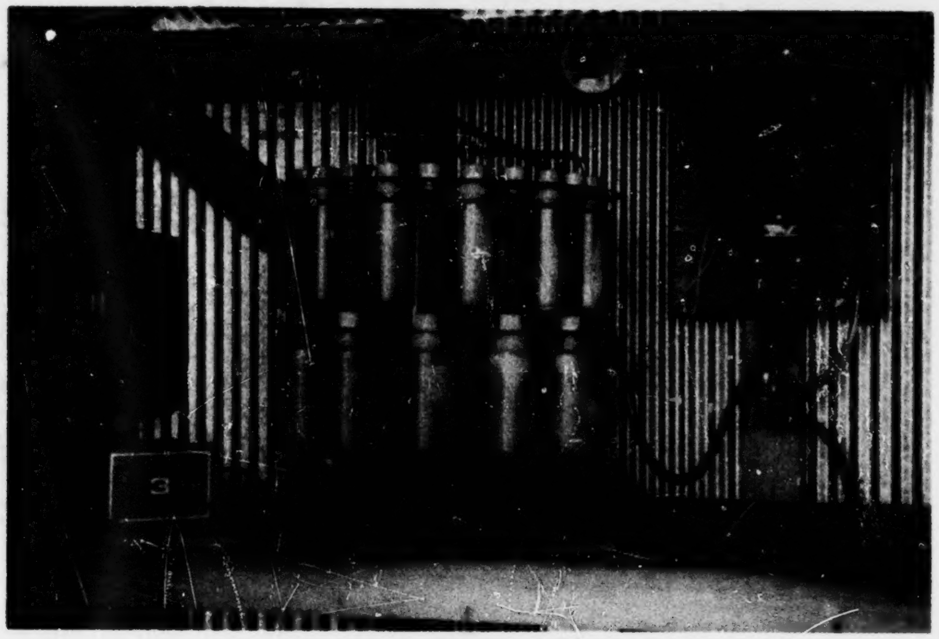


Figure 46 .
OUTLET MONITORING STATION
SEWAGE SAMPLING INSTALLATION
(SENTRY SEQUENTIAL EFFLUENT SAMPLER AND PUMP)

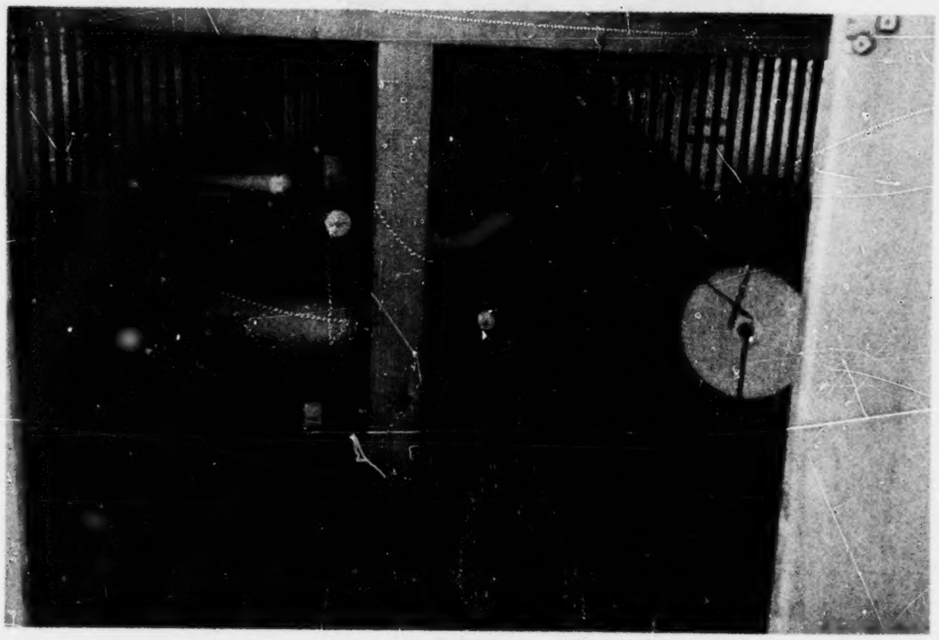


Figure 47 .
OUTLET MONITORING STATION
FLOW MEASURING INSTALLATION
(AIR-COMPRESSOR, MANOMETER, AND FLOW RECORDER)

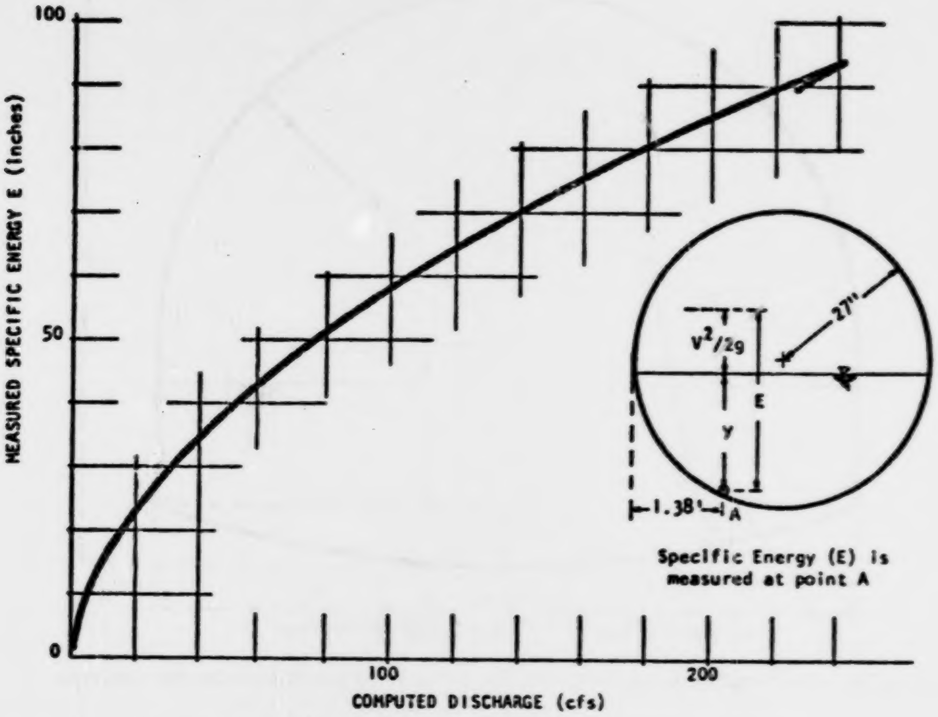


Figure 48. MEASURED SPECIFIC ENERGY VERSUS COMPUTED DISCHARGE FOR 54" CIRCULAR SEWER WITH SLOPE 0.013 (BANK)

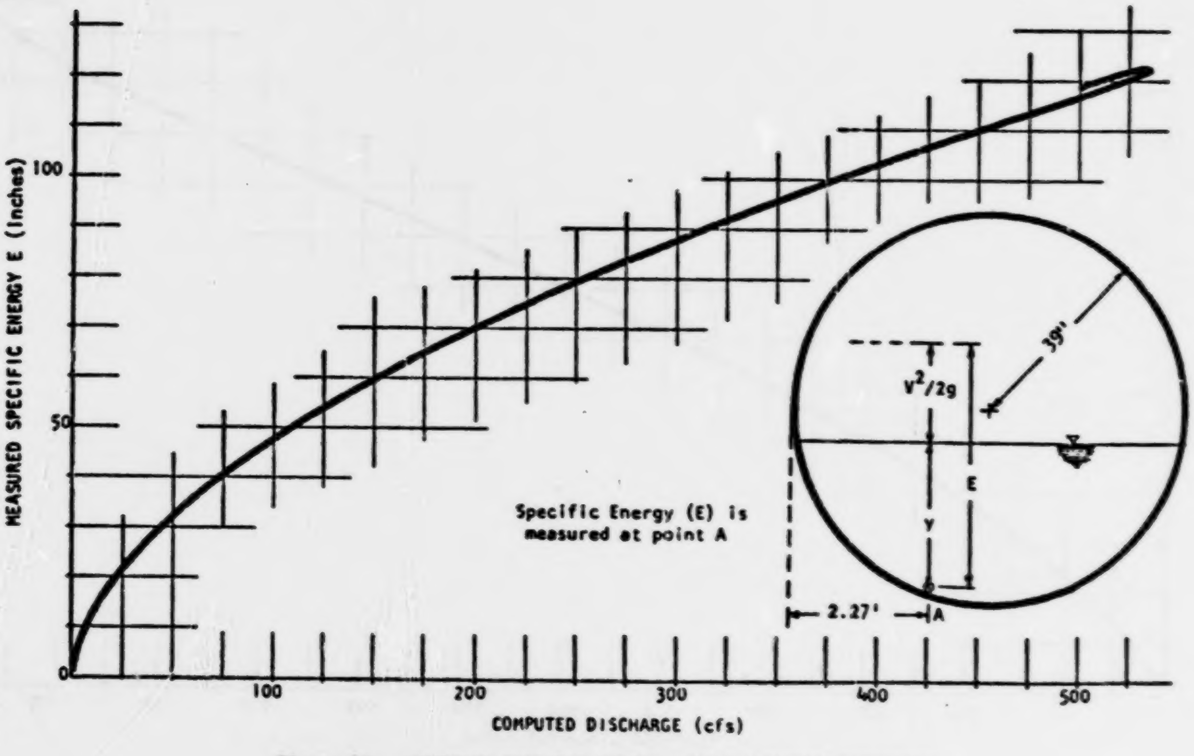
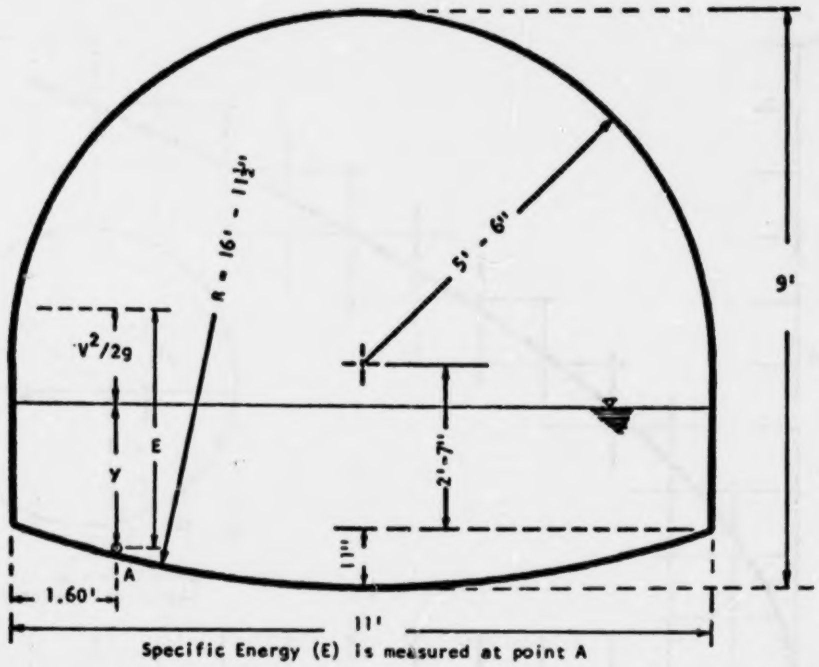


Figure 49. MEASURED SPECIFIC ENERGY VERSUS COMPUTED DISCHARGE FOR 78" CIRCULAR SEWER WITH SLOPE 0.009 (BANK)



V²/2g
E
Y
A
R = 16' - 11½"
5' - 6"
9'
2'-7"
11"
1.60'
11'

Specific Energy (E) is measured at point A

Figure 50. SEMICIRCULAR SEWER CROSS-SECTION 11' x 9' AT MONITORING STATION LONGVIEW

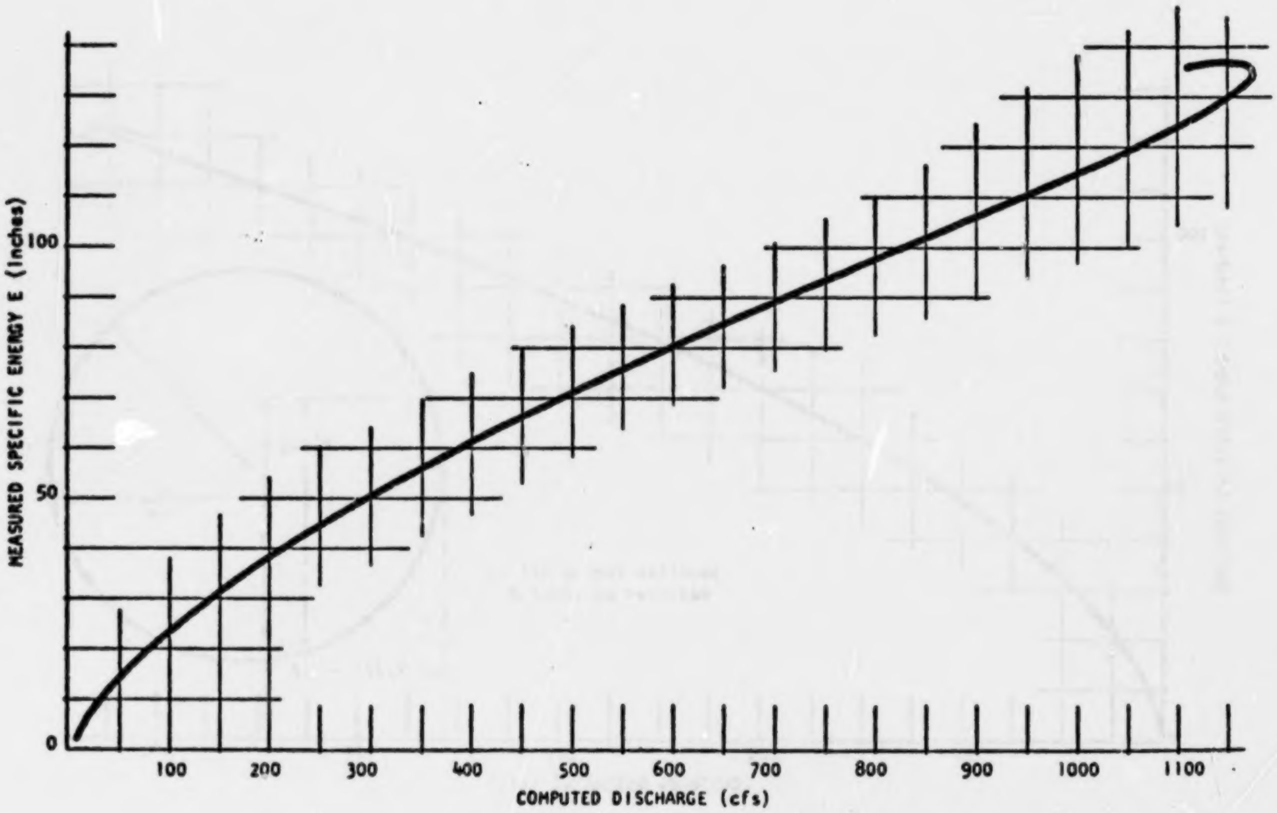
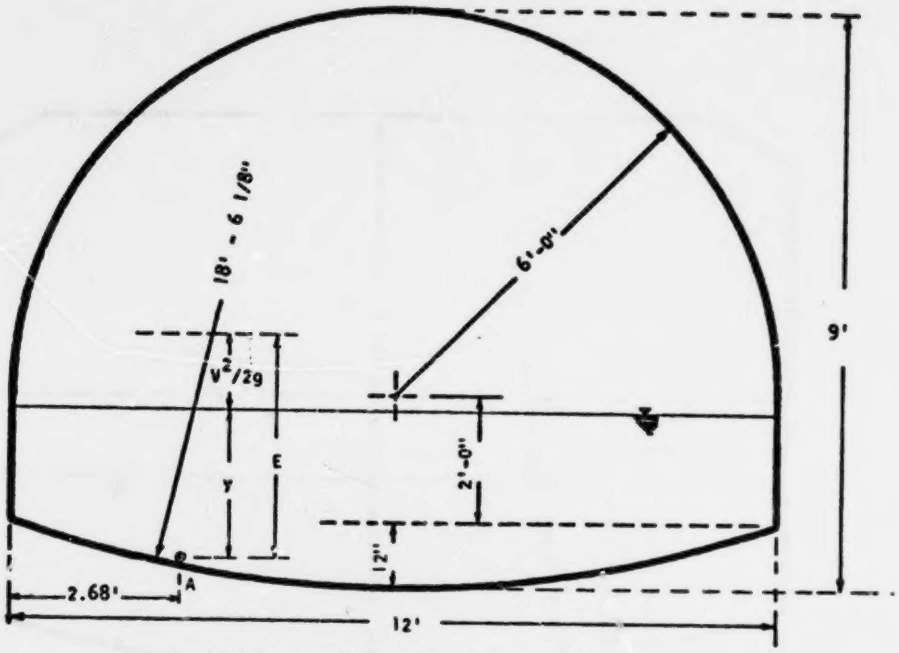


Figure 51. MEASURED SPECIFIC ENERGY VERSUS COMPUTED DISCHARGE FOR 11' x 9' SEMI-CIRCULAR SEWER WITH SLOPE 0.004 (LONGVIEW)


18' - 6 1/8"
6'-0"
9'
$V^2/2g$
Y
E
2'-0"
12"
2.68'
A
12'

Specific Energy (E) is measured at point A

Figure 52. SEMICIRCULAR SEWER CROSS-SECTION 12' × 9' AT MONITORING STATION LONGVIEW

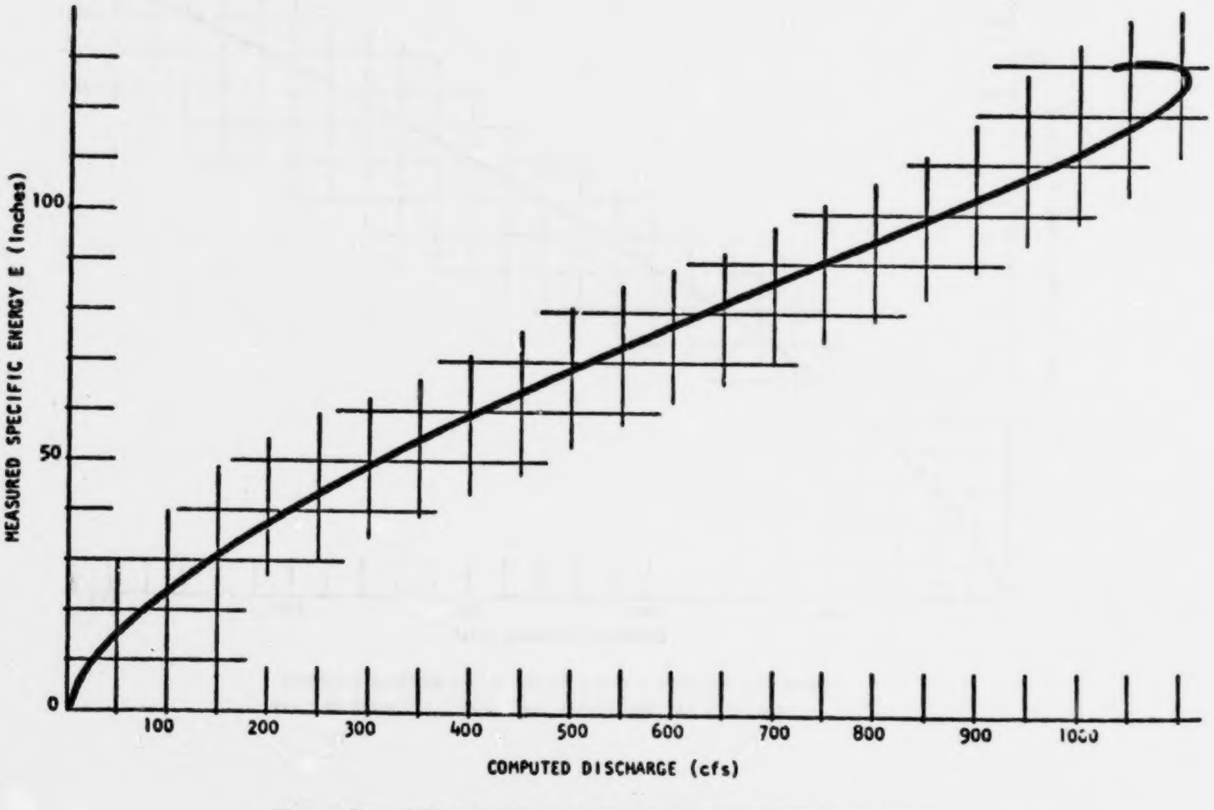
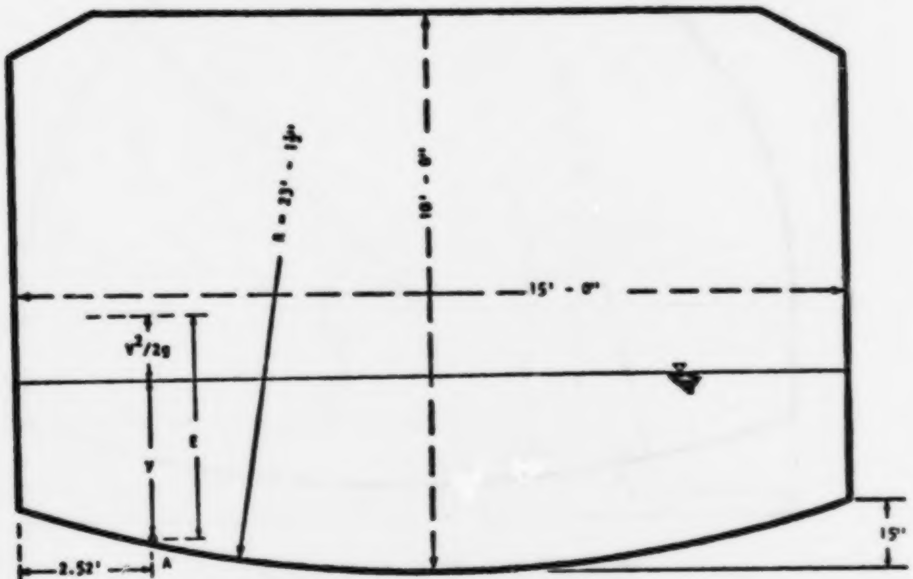


Figure 53. MEASURED SPECIFIC ENERGY VERSUS COMPUTED DISCHARGE
FOR 12' × 9' SEMI-CIRCULAR SEWER WITH SLOPE 0.003 (LONGVIEW)


R = 23' - 1½"
10' - 0"
15' - 0"
$V^2/2g$
E
Y
15"
2.52'
A

Specific Energy (E) is measured at point A

Figure 54. RECTANGULAR SEWER CROSS-SECTION 15' x 10' AT MONITORING STATION OUTLET

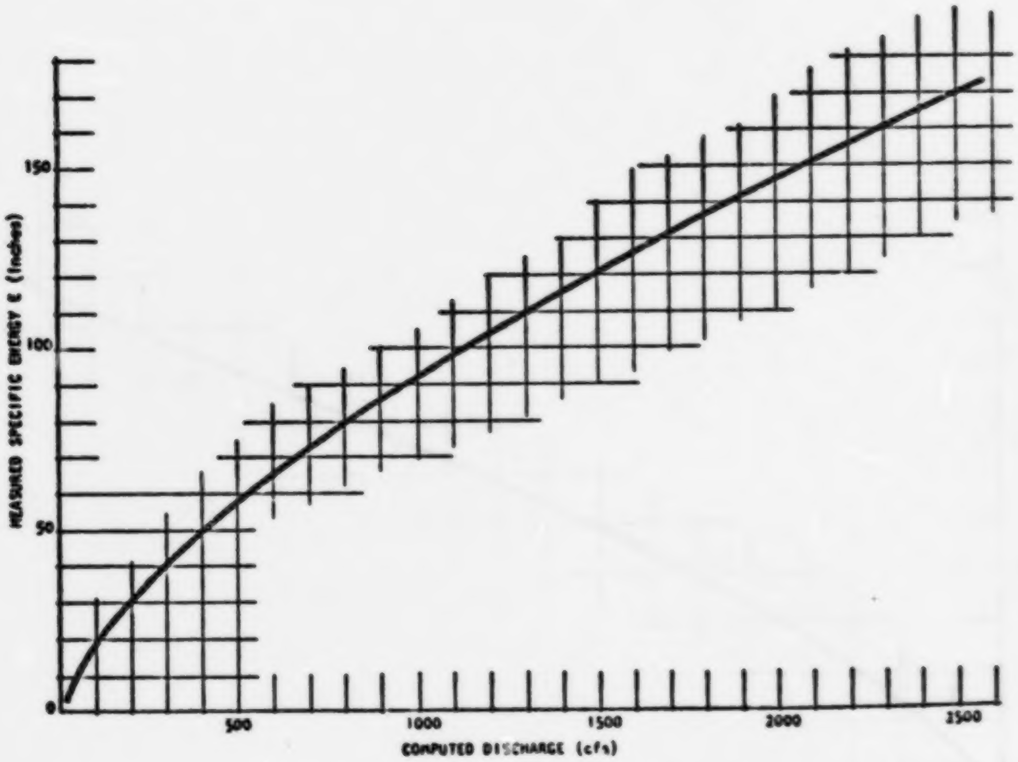


Figure 55. MEASURED SPECIFIC ENERGY VERSUS COMPUTED DISCHARGE FOR 10' x 15' RECTANGULAR SEWER WITH SLOPE 0.003 (OUTLET)

SECTION V

RAINFALL AND RUNOFF QUANTITY AND QUALITY DATA

1. Rainfall Data from U. S. Department of Commerce

The U. S. Department of Commerce has four raingages surrounding the Bloody Run Sewer Watershed. Their locations are shown on Figure 56.

Figure 57 is a topographic map of Hamilton County which can be used to interpret the variations in precipitation between the four above mentioned raingages.

On Table 8, the hourly precipitation in inches is tabulated from March, 1970 through October, 1970 for the Abbe Observatory raingage and the Greater Cincinnati Airport raingage.

On Table 9 the daily rainfall in inches is tabulated from March, 1970 through October, 1970 for the Abbe Observatory raingage, the Greater Cincinnati Airport raingage, the Cherry Grove raingage, and the West Fork Dam raingage. These data are also plotted on Figure 58 for convenience.

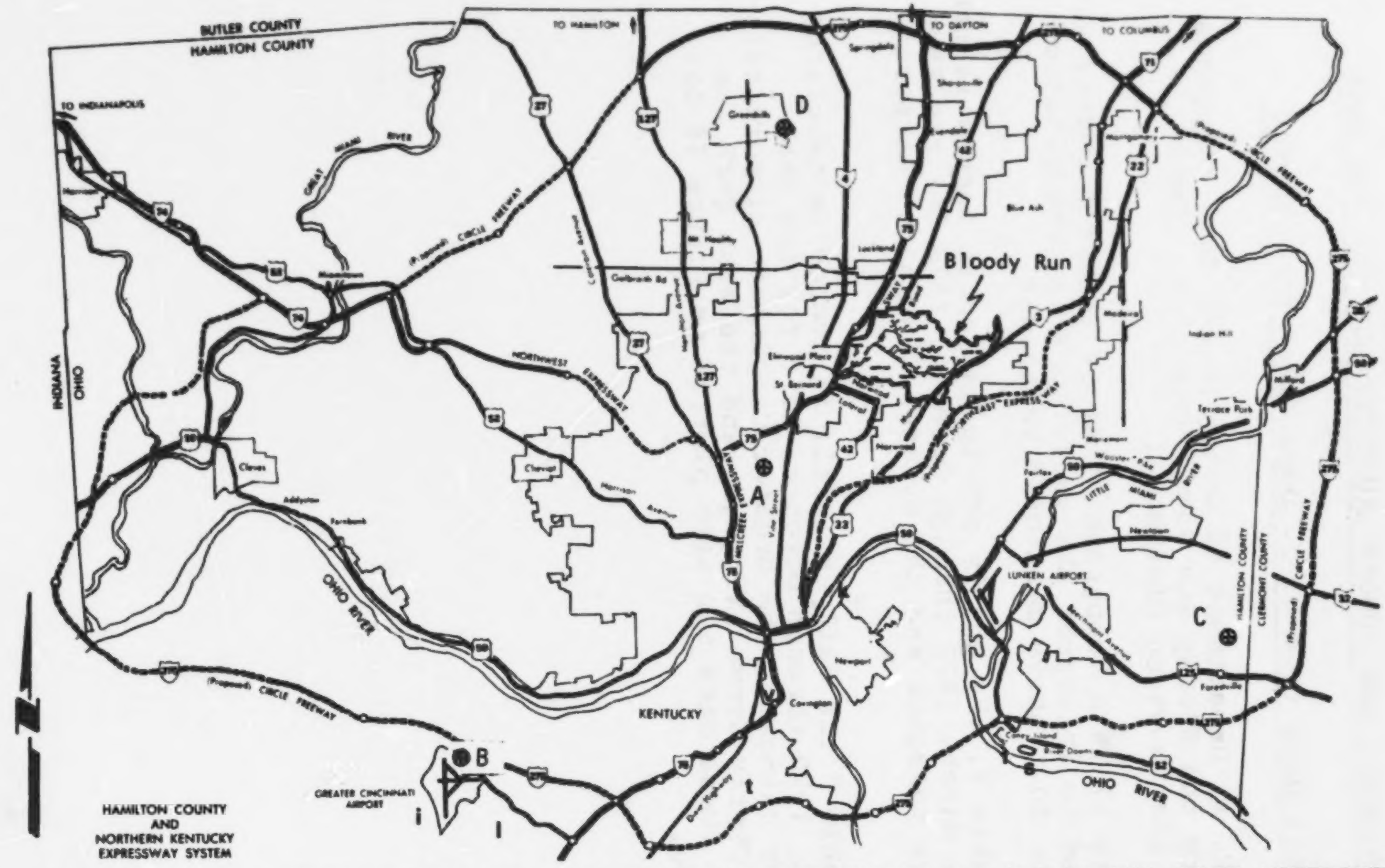

BUTLER COUNTY
HAMILTON COUNTY
TO INDIANAPOLIS
TO HAMILTON
TO DAYTON
TO COLUMBUS
INDIANA
OHIO
GREAT MIAMI RIVER
(Proposed) CIRCLE FREEWAY
NORTHWEST EXPRESSWAY
D
Bloody Run
A
C
B
OHIO RIVER
KENTUCKY
Newport
Covington
LUNKEN AIRPORT
LITTLE MIAMI RIVER
Newtown
Indian Hill
Blue Ash
HAMILTON COUNTY
CLERMONT COUNTY
GREATER CINCINNATI AIRPORT
HAMILTON COUNTY AND NORTHERN KENTUCKY EXPRESSWAY SYSTEM
N

Figure 56 . U. S. WEATHER BUREAU RAINGAGE STATIONS SURROUNDING THE BLOODY RUN SEWER WATERSHED

A: ABBE OBSERVATORY
B: GREATER CINCINNATI AIRPORT
C: CHERRY GROVE
D: WEST FORK DAM

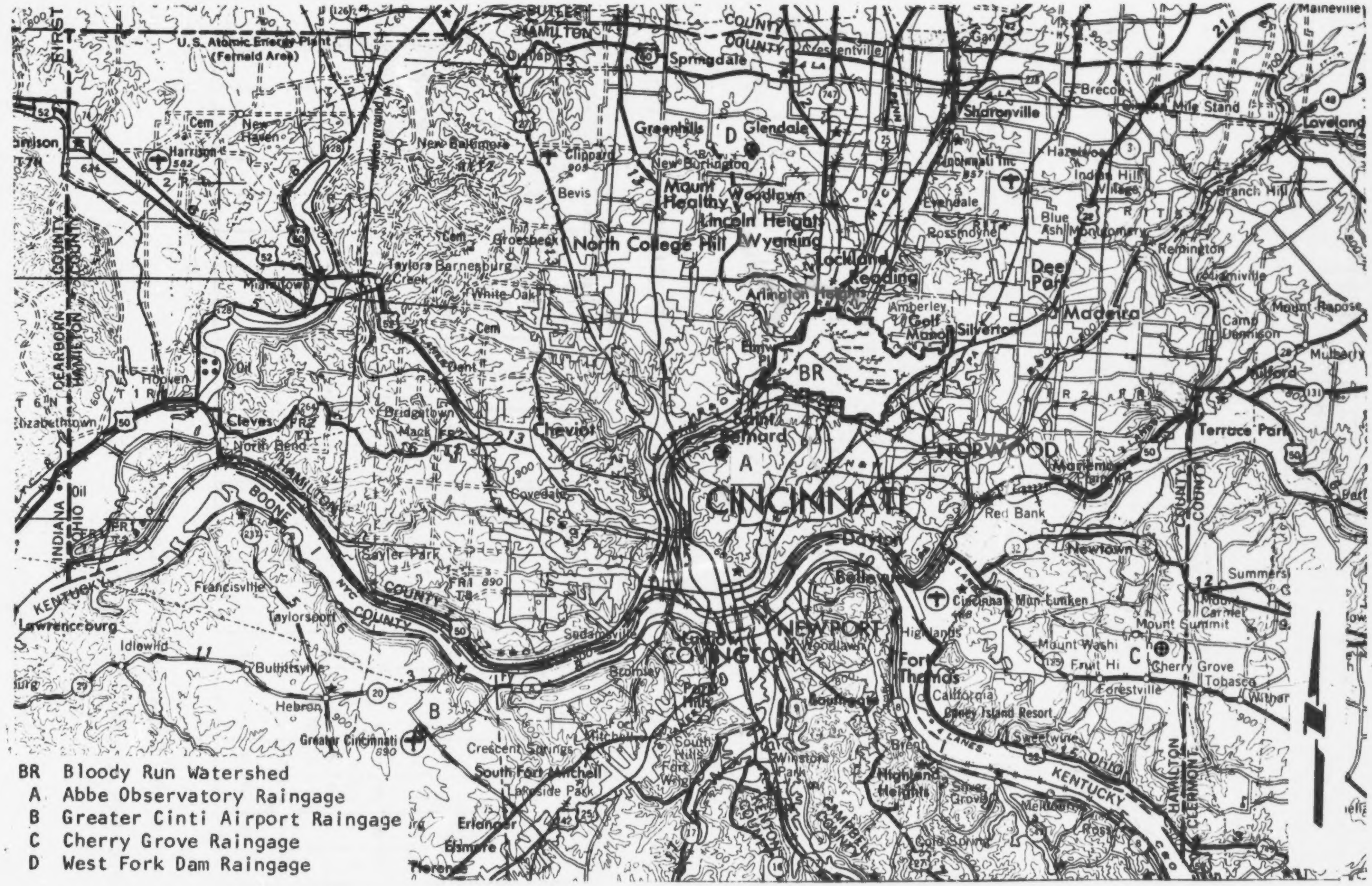

BR Bloody Run Watershed
A Abbe Observatory Raingage
B Greater Cinti Airport Raingage
C Cherry Grove Raingage
D West Fork Dam Raingage
CINCINNATI
NORWOOD
NEWPORT
COVINGTON
BR
A
B
C
D
Springdale
Sharonville
Greenhills
Glendale
Mount Healthy
Woodlawn
Lincoln Heights
Wyoming
North College Hill
Lockland
Reading
Deer Park
Madeira
Cheviot
Cleves
North Bend
Harrison
Loveland
Terrace Park
Milford
Newtown
Red Bank
Fort Thomas
Highland Heights
Lawrenceburg
Francisville
Taylorsport
Hebron
Bullittsville
Idlewild
Erlanger
Elsmere
Florence
Crescent Springs
South Fort Mitchell
Bellevue
Dayton
Cherry Grove
Mount Washington
Mount Carmel
Mount Summit
Forestville
Summerside
Tobasco
Silverton
Amberley
Arlington Heights
Elmwood
Saint Bernard
Mariemont
Camp Dennison
Remington
Indian Hill
Blue Ash
Montgomery
Evendale
Rossmoyne
Hazelwood
Brecon
Twenty Mile Stand
Branch Hill
Mount Repose
Mulberry
Bevis
Clippard
New Baltimore
New Burlington
Groesbeck
White Oak
Taylors Creek
Miamitown
New Haven
Hooven
Elizabethtown
Bridgetown
Sayler Park
Covedale
Sedamsville
Bromley
Greater Cincinnati
Cincinnati Muni-Lunken
California
Coney Island Resort
U.S. Atomic Energy Plant (Fernald Area)
BUTLER COUNTY
HAMILTON COUNTY
DEARBORN COUNTY
HAMILTON COUNTY
CLERMONT COUNTY
HAMILTON COUNTY
BOONE COUNTY
KENTON COUNTY
CAMPBELL COUNTY
INDIANA
OHIO
KENTUCKY
OHIO
Maineville
Lakeside Park

Figure 57. TOPOGRAPHIC MAP OF HAMILTON COUNTY (SCALE 1:250,000)

Table 8 . HOURLY PRECIPITATION*
MARCH 1970

CINCINNATI, OHIO - ABBE OBSERVATORY STATION

HOURLY PRECIPITATION (Water equivalent in inches)

Date	A. M. Hour ending at												P. M. Hour ending at												Date
	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	
1																T					T	.02	T		1
2								.12	.18	.07	.01	.03	.02	.06	.04	.01		T	T	.06	.14	.41	.01	T	2
3	.03	T	.01	.05	.08	.09	.04	.08	.06	.03	T	T		T	T					T	.02	.01	.05	.04	3
4								T										.26	T	T	T	T	T	T	4
5																									5
6																									6
7																									7
8																									8
9																									9
10										.01	.01	.02	T												10
11																		T							11
12					T	T	.01	.01	.02	.01	.03	.03	.03	.03	.04	.04	.04	.02	.01	.02	.02	.02	.01	.01	12
13	T	.01	T																						13
14		T	T	T							.01	T	T	T	T								T	.01	14
15	T	T						.01	.01	T	.01	.01			T	T	.01								15
16																									16
17												T	T	T	T	T	T	.05	.07	.08	.08	.12	.10	.10	17
18	.07	.01	.01	.01	T	T	T								T	T	T								18
19																								T	19
20	T	T	.01	T	T			T	T																20
21																									21
22	T	T	T	.02	.03	.03	.02	.03	.01	T	T	.01	T	T	.04	.02	T								22
23																									23
24						T	T																		24
25												T	.18	.15	.27	.10	.01	.01	T						25
26		.04	.15	.10	T	.01	.02																		26
27																									27
28																									28
29	T	.01	.02	.02	.02	.02	.01	.02	.02	.01															29
30																								T	30
31	T	T	T	T																					31

BOONE COUNTY, KENTUCKY - GREATER CINCINNATI AIRPORT STATION

HOURLY PRECIPITATION (Water equivalent in inches)

Date	A. M. Hour ending at												P. M. Hour ending at												Date
	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	
1																					T	.02	T		1
2				T	T			.14	.24	.04	.02	.06	.07	.09	T	T		T	.04	T	.14	.02	T		2
3	.01	T	.01	.08	.10	.06	.05	.04	.08	.01	T	T	T	T	T			T	T	T	.03	.02	.05	.05	3
4				T	T	T		T	T	T							.10	.18	.02	T	T	T	T		4
5																									5
6																									6
7																									7
8																									8
9																									9
10									.01	T	T	T	T												10
11																		T							11
12			T	T	.01	.01	T	T	T	T	T	.01	.01	.04	.14	.11	.02	.01	.01	.04	.03	.02	.03	.02	12
13	.01	T	T																T	T		T	T	T	13
14	T	T	.01	T		T	T				T	T	T	T	T	T		T	T			T	T	T	14
15	.01	T	T	T	T	T	T	T	.02	.02	.01	T		T	T		T	T	T	T	T				15
16																									16
17										T	T	T	T	T	T		.02	.04	.10	.12	.14	.12	.07	.01	17
18	.04	.03	.02	.01	T	T		T	T	T				T	T	T	T	T	T	T					18
19																								T	19
20	T	.01	.01	.01	T			T	T																20
21																								T	21
22	T	T	.02	.02	.03	.03	.03	.03	.01	T	T	T	T	.01	.05	.01	.01								22
23																									23
24				T	T	T	T	T	T	T	T	T	T												24
25											T	.05	.14	.17	.23	.07	.01	.02	T						25
26		.05	.15	.10	T	.05	T	T								T	T								26
27																									27
28																									28
29	T	.05	.04	.10	.03	.02	.02	T	.02	T															29
30																								T	30
31	T	T	T	T																					31

*Data from the Department of Commerce

**T indicates trace

Table 8 . HOURLY PRECIPITATION*
APRIL 1970

CINCINNATI, OHIO - ABBE OBSERVATORY STATION

HOURLY PRECIPITATION (Water equivalent in inches)

Date	A. M. Hour ending at												P. M. Hour ending at												Date
	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	
1				T	T										T	.13	.15	.02	.05	.27	.13	.23	.22	.34	1
2	.28	.27	.27	.05	.01	T	.01	.01	T	T	T	T	T	T	T	T									2
3																									3
4							T	T	.01	T	.01	T													4
5																									5
6								.02	.09			T		T	.01	T	T	T	T	T					6
7																									7
8																									8
9																									9
10																									10
11																									11
12																									12
13						.13	.09	.02						T	T										13
14					T	.01	T	T																	14
15																									15
16																									16
17																									17
18																					T	.03	T		18
19					.01	.45	.12	T	.02	T								.02	.02	.03	.03	.09	.16	.01	19
20	.03	T													T	T								T	20
21	.03	.01																							21
22			T	T	T								T	T										T	22
23	.05	.03		.43	T	.05	.02				T	.01	T	T	T					.02	.03	T	.19	.02	23
24	.16	.01	.46	.18	.11	.15	.10			.04	.02	.04	.01												24
25																						.03	.12	.01	25
26			.05																						26
27																.02	.19	.11	.01						27
28						.03	T					.03	T										T	.01	28
29	T	T	T																.05	T	.13	.02	T		29
30																									30

BOONE COUNTY, KENTUCKY - GREATER CINCINNATI AIRPORT STATION

HOURLY PRECIPITATION (Water equivalent in inches)

Date	A. M. Hour ending at												P. M. Hour ending at												Date
	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	
1				T											.02	.14	.15	.03	.03	.25	.11	.29	.23	.33	1
2	.27	[illegible]	.19	.03		.01	T	T		T	T	T	T	T	T										2
3																									3
4							T	T	.01	.01	T	T													4
5																									5
6								.03	.08			T	T			T	T	T	T	T					6
7																									7
8																									8
9																									9
10																									10
11																									11
12																				T	T				12
13						.08	.09	.04	T						T	T								T	13
14	T	T		.01	.03	T	T	T	T	T	T														14
15																									15
16											T	T													16
17																									17
18																					T	.01			18
19					.04	.29	.11	.01	T	T							T	.01	.04	T	.02	.14	.13	T	19
20	.01	T	T	T												T								T	20
21	.01	T																							21
22			T							T			T								T	T		T	22
23	.08	.03	T	.60	.01		.02				T	.01	.01	T	T		T	T		.02	.03		.08	[illegible]	23
24	.45	T	.20	.17	.10	.18	.08	T		.01	.02	.05	T												24
25																					T	.03	.05	.05	25
26	.02	T																							26
27															T	.15	.18	.07	T						27
28						.05	T				T	T											T	T	28
29	T	T																.14	.27	.01	.32	.05			29
30																									30

*Data from the Department of Commerce

**T indicates trace

Table 8 . HOURLY PRECIPITATION*
MAY 1970

CINCINNATI, OHIO - ABBE OBSERVATORY STATION

HOURLY PRECIPITATION (Water equivalent in inches)

Date	A.M. Hour ending at												P.M. Hour ending at												Date
	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	
1																.08	T			.01	T	.01	.04		1
2	.02	.01			.02	.02	.03	.02	.01	T															2
3																									3
4																									4
5																	T								5
6																									6
7																									7
8																									8
9																									9
10											T	T	.01												10
11																				.09	.01	T			11
12																		.02	.61	.11					12
13													T		T	T									13
14																						T			14
15																									15
16	.10	T	.03									T	.02	T							.09	T			16
17								T	T					T											17
18																									18
19																									19
20																									20
21																									21
22																									22
23																									23
24		.02																							24
25		.01														.03	.04	.05	.19	.10	T	T			25
26																									26
27																									27
28																									28
29																									29
30																									30
31																									31

BOONE COUNTY, KENTUCKY - GREATER CINCINNATI AIRPORT STATION

HOURLY PRECIPITATION (Water equivalent in inches)

Date	A.M. Hour ending at												P.M. Hour ending at												Date
	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	
1															T	.20				T	T	.01	.02		1
2	.01	.01	T	T	.01	.02	.01	.03	.01	T															2
3																									3
4																									4
5																	T								5
6																									6
7																									7
8																									8
9																									9
10											T	.01	T												10
11																				.13	.03				11
12																	T			.01	T				12
13											-		.01	T	.01										13
14																						.02	T		14
15																							T	.10	15
16	.10								T			T	.01							.21	.07	.01	T		16
17								T	T					T											17
18																									18
19																									19
20																									20
21																									21
22																									22
23																									23
24																									24
25															T	.02	T	.38	.01	.62					25
26																									26
27																									27
28																									28
29											T	T													29
30																									30
31	T							T	T	T	T														31

*Data from the Department of Commerce

**T indicates trace

Table 8 . HOURLY PRECIPITATION*
JUNE 1970

CINCINNATI, OHIO - ABBE OBSERVATORY STATION

HOURLY PRECIPITATION (Water equivalent in inches)

Date	A. M. Hour ending at												P. M. Hour ending at												Date
	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	
1			T	T	T											.01	.20	.01							1
2													T									.40	.14	T	2
3												.01	.09	.03	.01	T	.01	.02	T	.01	.01	.02	.02	.02	3
4	.02	.02	.09	.14	.14	.07	.03	T	.01	T													.06	1.04	4
5	.16	.01	.01									.01									.21	.02	.20	.04	5
6							T	T	T																6
7																									7
8																									8
9																									9
10																									10
11																									11
12																									12
13							.03	.16	.09	T															13
14																									14
15			.16	T																					15
16															T										16
17																									17
18																									18
19														T											19
20																									20
21						.13																			21
22																									22
23																									23
24																									24
25						.01						T													25
26												.16				.01	.80	T		.31	.01				26
27																									27
28																									28
29																									29
30																									30

BOONE COUNTY, KENTUCKY - GREATER CINCINNATI AIRPORT STATION

HOURLY PRECIPITATION (Water equivalent in inches)

Date	A. M. Hour ending at												P. M. Hour ending at												Date
	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	
1	T	T				T										T	.41	T							1
2																	.05	T			.05	.36	.12	.01	2
3	T	T	T	T	T	T						.04	.37	.03	T	T	.01	.01	T	.01	.02	.02	.03	.02	3
4	.01	.03	.01	.05	.08	.04	T	T	T	T													.38	.87	4
5	.13	.01											T		T					T	.33	.01	.04	.05	5
6	T					.01																			6
7																									7
8																									8
9																									9
10																									10
11																									11
12																									12
13							.02	.28	1.23	T															13
14																									14
15		.01	.01	T																					15
16														T	T										16
17																									17
18																									18
19														T											19
20																									20
21						.01																			21
22																									22
23																									23
24																									24
25					T	T																			25
26												T				.06	.45	T		T	T	.02			26
27																									27
28																									28
29																									29
30																									30

*Data from the Department of Commerce

**T indicates trace

Table 8 . HOURLY PRECIPITATION*
JULY 1970

CINCINNATI, OHIO - ABBE OBSERVATORY STATION

HOURLY PRECIPITATION (Water equivalent in inches)

Date	A.M. Hour ending at												P.M. Hour ending at												Date
	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	
1																									1
2																									2
3										T	.01														3
4	T																								4
5																									5
6																									6
7																									7
8					T								.01	T	.36	.02	.70	T				T	.14		8
9													.02	.04			.27	.11							9
10																									10
11																									11
12																									12
13																									13
14																									14
15																									15
16																									16
17																									17
18																									18
19				T	T	T											.01	.01	T	.01	.13	.04		T	19
20	.02					.06	1.02			T															20
21																									21
22																			T	T	T	T	.01	.01	22
23	T	.02	.02	T	T	.04	.02	T																	23
24																									24
25																									25
26													.01	T	T	T	.01	T	T	T					26
27					.01												.01	.04	T						27
28																									28
29																									29
30												.01													30
31				.01			.01																		31

BOONE COUNTY, KENTUCKY - GREATER CINCINNATI AIRPORT STATION

HOURLY PRECIPITATION (Water equivalent in inches)

Date	A.M. Hour ending at												P.M. Hour ending at												Date
	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	
1																									1
2															T										2
3										.02	.01														3
4	T																								4
5																									5
6																									6
7																									7
8														T	.59	.05	1.17	.39					.05		8
9													T	T			.02	T				T	T		9
10																									10
11																									11
12																									12
13																									13
14																									14
15																		T							15
16																									16
17																									17
18																									18
19					T	T											.02		T	.02	.09	.04			19
20	.23					.01	.53	T	T																20
21																									21
22																				T	T	T	.01	.03	22
23	T	.02	.03	.01	.01	.01	.01	T																	23
24																									24
25																									25
26												T	T	T	T	T	T	T	T	T					26
27														T	T		T	.05	T						27
28																									28
29																									29
30												T													30
31		T	.01	.01		.01	.04	T	T																31

*Data from the Department of Commerce

**T indicates trace

Table 8 . HOURLY PRECIPITATION*
AUGUST 1970

CINCINNATI, OHIO - ABBE OBSERVATORY STATION

HOURLY PRECIPITATION (Water equivalent in inches)

Date	A. M. Hour ending at												P. M. Hour ending at												Date
	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	
1				.18	.10																				1
2																									2
3								.08	.05	.04	.11	.02		.18											3
4																									4
5																									5
6																									6
7																									7
8																		T	T	T	.01	T		.05	8
9	.01			.08	.10	.01	T	T	T	.01	.01	T	.09	.50	.18	.13	.17	.31	.05	.03	.07	.01	.04	.02	9
10	.02	T	T	T		.01	.03	.02	T	T															10
11																									11
12														.25	.03										12
13																	T	.45							13
14																									14
15																									15
16																T	.01								16
17																									17
18																									18
19			T	T										.04	.01	T									19
20	T	T	T																	T					20
21																									21
22																.01	.80	.02					.01	T	22
23																									23
24																									24
25																									25
26																									26
27																									27
28																									28
29																									29
30													T	T											30
31																									31

BOONE COUNTY, KENTUCKY - GREATER CINCINNATI AIRPORT STATION

HOURLY PRECIPITATION (Water equivalent in inches)

Date	A. M. Hour ending at												P. M. Hour ending at												Date
	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	
1																									1
2																									2
3						T	T			.01	.10	.01		.24	T										3
4																									4
5																									5
6																									6
7																									7
8																		T	.01	.02	.01	T	T	T	8
9	T	T	.01	.24	.12	T	.01	T	.08	.01	.01	T	T	.03	T	.06	T		T	T	.31	.04	.14	.07	9
10	.01	T	T	T	T	T	T	T	T																10
11																									11
12												.02	T			.12	.02								12
13															.02										13
14																									14
15																									15
16																	T	T							16
17							T	.02		T															17
18																									18
19			T										T	.40	.02	T	T						T	T	19
20	T	T	T																.01	.42					20
21																									21
22																.05	.55	T					T	T	22
23					T	T	T	T																	23
24																									24
25																									25
26																									26
27																									27
28																									28
29																									29
30												T	.10	.07											30
31																									31

*Data from the Department of Commerce

**T indicates trace

Table 8. HOURLY PRECIPITATION*
SEPTEMBER 1970

CINCINNATI, OHIO - ABBE OBSERVATORY STATION

HOURLY PRECIPITATION (Water equivalent in inches)

Date	A. M. Hour ending at												P. M. Hour ending at												Date
	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	
1																									1
2																									2
3									T						T		T	T	.22	T		.01			3
4															T									.09	4
5	.05																								5
6																									6
7	.01	T						T	T																7
8																.29	.26								8
9																									9
10																									10
11																									11
12																									12
13																						.54	.01		13
14					.02																				14
15																									15
16																									16
17																									17
18											.02														18
19																									19
20																									20
21															T	T									21
22																					.46	.03			22
23																									23
24																		.30							24
25							.18	.39	.07	.01	T														25
26															.25	.02	.02	T	.01	T	.07	.08	.04	.02	26
27	.01	.02	.04	.01																					27
28																									28
29																									29
30																									30

BOONE COUNTY, KENTUCKY - GREATER CINCINNATI AIRPORT STATION

HOURLY PRECIPITATION (Water equivalent in inches)

Date	A. M. Hour ending at												P. M. Hour ending at												Date
	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	
1																									1
2																		T							2
3								T	.01						T		T	.05	.21		.03	T			3
4															T	T							.01	.13	4
5	.01	T																							5
6																				T	T			T	6
7	T	T					.01	.08	.02																7
8																T	.54								8
9																									9
10																									10
11																									11
12																									12
13																						.05	T		13
14					T													.01	.01						14
15																									15
16																									16
17																									17
18										.13	.05		T												18
19																									19
20																									20
21															T	.09									21
22																				T	.40	.01		.01	22
23	T																								23
24																	.13	.34							24
25							.44	.19	.09	T															25
26														T	.52	.04	.02	.01	T	.01	.04	.05	.03	.02	26
27	T	.03	.03	.02																					27
28																									28
29																									29
30																									30

*Date from the Department of Commerce

**T indicates trace

Table 8 . HOURLY PRECIPITATION*
OCTOBER 1970

CINCINNATI, OHIO - ABBE OBSERVATORY STATION

HOURLY PRECIPITATION (Water equivalent in inches)

Date	A.M. Hour ending at												P.M. Hour ending at												Date
	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	
1																									1
2																									2
3																									3
4																									4
5																									5
6																									6
7																									7
8												T	T	T	T			T	.06		.01	.01			8
9																		T			.01	T	.15	.18	9
10	.01	.03																							10
11																									11
12							.01			.03	.02	T	.02	.01	.01	.01		.01	.02	.04	.06	.03	.02		12
13	.05	.10	.01						T	T			T	T											13
14		.02	.09	.18	.07	.02	.01											T	.01	.03	.01				14
15																									15
16																									16
17																									17
18																									18
19																									19
20			.02	.08	.08	.18	.18	.08	.01	T	.03	.01	.03	.01	.04	.01	.04	.05	.02	.01	T	T	.01		20
21							T																		21
22																									22
23																									23
24																				T					24
25																									25
26																									26
27																									27
28											T			T	.01	T	.01	.01	T						28
29			.02	T	.02	.01	.02	.05	.02	T	T	.03	.01	.03	.03	.01	.01	T	T	.01				T	29
30	T		T	T																					30
31																									31

BOONE COUNTY, KENTUCKY - GREATER CINCINNATI AIRPORT

HOURLY PRECIPITATION (Water equivalent in inches)

Date	A.M. Hour ending at												P.M. Hour ending at												Date
	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	
1																									1
2																									2
3																									3
4																									4
5																									5
6																									6
7																									7
8												T	T				T	.03	T		.01	T	T		8
9														T						T	.04	.06	.10	.04	9
10	.04	.02																							10
11																									11
12							T	T	T	.02	.03	T	.03	.01	T	T		.01	.02	.04	.05	.04	.01	.01	12
13	.09	.10	.02	T					T	T	T	T	T												13
14	T	.02	.10	.16	.10	.05	T						T	T				.04	T	.01	T	T			14
15									T	T	T	T	.01	T	T										15
16																									16
17																									17
18																									18
19																									19
20	T	.02	.05	.07	.11	.17	.13	.02	T	T	.02	.02	.07	.01	.02	.07	.04	.03	.04	T	.01	.06	T	T	20
21	T			T	T		T	T																	21
22																									22
23																									23
24																				T					24
25																									25
26																									26
27																								T	27
28										T		T	T	T	T	T	.01	.01	T		T	T			28
29			.01	T	.01	.02	.01	.02	T	T	.01	.01	T	.02	.04	T		T						T	29
30	T		T	T																					30
31																									31

*Date from the Department of Commerce

**T indicates trace

Table 9 . DAILY RAINFALL DATA* (WATER EQUIVALENT IN INCHES) METROPOLITAN AREA OF CINCINNATI, OHIO

MARCH 1970

Date	Abbe Observatory	Greater Cincinnati Airport	Cherry Grove	West Fork Dam
1	0.02	0.02	0	0
2	1.16	0.86	0.52	0.02
3	0.12	0.15	0.43	0
4	0.73	0.74	0.57	0.50
5	T	T	0.04	0.42
6	0	0	0	0
7	0	0	0	0
8	0	0	0	0
9	0	0	0	0
10	0.04	0.01	0.08	0
11	T	T	0	0.04
12	0.40	0.51	0.34	T
13	0.01	0.01	T	0.25
14	0.02	0.01	0.17	T
15	0.04	0.06	0.10	0.06
16	0	0	T	T
17	0.06	0.72	0.30	0
18	0.10	0.10	0.28	0.70
19	T	T	0	T
20	0.01	0.03	0	T
21	0	T	0	0
22	0.21	0.25	0.26	0.05
23	0	0	T	0.15
24	T	T	T	T
25	0.72	0.69	0.70	0
26	0.32	0.35	0.17	1.00
27	0	0	0	T
28	0	0	0	0
29	0.15	0.30	0.29	0.14
30	T	T	0	T
31	T	T	0	0

T Designates trace

* Records from U. S. Department of Commerce

Table 9 . DAILY RAINFALL DATA* (WATER EQUIVALENT IN INCHES) METROPOLITAN AREA OF CINCINNATI, OHIO

APRIL 1970

Date	Abbe Observatory	Greater Cincinnati Airport	Cherry Grove	West Fork Dam
1	1.54	1.53	0.45	0.04
2	0.90	0.86	2.09	2.45
3	0	0	0	T
4	0.02	0.02	T	0
5	0	0	0	0.05
6	0.12	0.13	0.08	0
7	0	0	0	0.15
8	0	0	0	0
9	0	0	0	0
10	0	0	0	0
11	0	0	0	0
12	0	T	0	0
13	0.24	0.21	0.07	0.18
14	0.01	0.04	0	T
15	0	0	0	0
16	0	T	0	0
17	0	0	0	0
18	0.03	0.01	0	0
19	0.96	0.81	0.45	0.35
20	0.03	0.01	0.09	0.40
21	0.04	0.01	0	0.05
22	T	T	0	0.04
23	0.87	0.91	0.88	0.11
24	1.26	1.26	1.78	1.25
25	0.16	0.13	0	0.07
26	0.05	0.02	0.11	0.20
27	0.33	0.40	0.12	0
28	0.07	0.05	0.04	0.10
29	0.20	0.79	0	0.40
30	0	0	0.10	T

T Designates trace

* Records from U. S. Department of Commerce

Table 9 . DAILY RAINFALL DATA* (WATER EQUIVALENT IN INCHES) METROPOLITAN AREA OF CINCINNATI, OHIO

MAY 1970

Date	Abbe Observatory	Greater Cincinnati Airport	Cherry Grove	West Fork Dam
1	0.14	0.23	0	0
2	0.13	0.10	0.22	T
3	0	0	0	0.05
4	0	0	0	0
5	T	T	0	0
6	0	0	0	0
7	0	0	0	0
8	0	0	0	0
9	0	0	0	0
10	0.01	0.01	0.05	0
11	0.10	0.16	0	T
12	0.74	0.01	T	0.15
13	T	0.02	0.94	1.13
14	T	0.02	T	0.25
15	0	0.10	0.12	0
16	0.22	0.20	T	0.25
17	T	T	0.09	0.10
18	0	0	0	0
19	0	0	0	0
20	0	0	0	0
21	0	0	0	0
22	0	0	0	0
23	0	0	0	0
24	0.62	0	0	0.02
25	0.42	1.03	T	0.10
26	0	0	0.44	0.65
27	0	0	0	0
28	0	0	0	0
29	0	T	0	0
30	0	0	0.07	0
31	0	T	0	0

T Designates trace

* Records from U. S. Department of Commerce

Table 9 . DAILY RAINFALL DATA* (WATER EQUIVALENT IN INCHES) METROPOLITAN AREA OF CINCINNATI, OHIO

JUNE 1970

Date	Abbe Observatory	Greater Cincinnati Airport	Cherry Grove	West Fork Dam
1	0.22	0.41	0	0
2	0.54	0.59	T	0.50
3	0.25	0.56	1.83	0.50
4	1.64	1.47	0.44	0.80
5	0.66	0.60	0.74	0.87
6	T	0.01	0.11	0.50
7	0	0	0	0
8	0	0	0	0
9	0	0	0	0
10	0	0	0	0
11	0	0	0	0
12	0	0	0	0
13	0.28	1.53	0.14	0
14	0	0	0	0.40
15	0.16	0.02	0.63	T
16	T	T	0	0
17	0	0	0	0
18	0	0	0	0
19	T	T	0	0
20	0	0	0	0
21	0.13	0.01	0.15	T
22	0	0	0	0
23	0	0	0	0
24	0	0	0	0
25	0.01	T	0	0
26	1.29	0.53	1.14	0
27	0	0	0.28	0.43
28	0	0	0	0
29	0	0	0	0
30	0	0	0	0

T Designates trace

* Records from U. S. Department of Commerce

Table 9 . DAILY RAINFALL DATA* (WATER EQUIVALENT IN INCHES) METROPOLITAN AREA OF CINCINNATI, OHIO

JULY 1970

Date	Abbe Observatory	Greater Cincinnati Airport	Cherry Grove	West Fork Dam
1	0	0	0	0
2	0	T	0.20	0
3	0.01	0.03	0	0
4	T	0	0.08	T
5	0	0	0	0
6	0	0	0	0
7	0	0	0	0
8	1.23	2.25	1.70	0
9	0.44	0.02	0.25	1.30
10	0	0	0	0.65
11	0	0	0	0
12	0	0	T	0
13	0	0	0.22	0
14	0	0	0	0
15	0	T	0.12	0
16	0	0	0.04	T
17	0	0	0	0
18	0	0	0	0
19	0.20	0.17	T	0
20	1.10	0.75	0.42	0.05
21	0	0	0	0.95
22	0.02	0.04	T	0
23	0.10	0.09	0.14	0.05
24	0	0	0	0.05
25	0	0	0	0
26	0.02	T	0.09	0
27	0.06	0.05	0.11	T
28	0	0	0.10	0.50
29	0	0	0	0
30	0.01	T	0.05	0
31	0.02	0.07	T	0.15

T Designates trace

* Records from U. S. Department of Commerce

Table 9. DAILY RAINFALL DATA* (WATER EQUIVALENT IN INCHES) METROPOLITAN AREA OF CINCINNATI, OHIO

AUGUST 1970

Date	Abbe Observatory	Greater Cincinnati Airport	Cherry Grove	West Fork Dam
1	0.28	0	0	T
2	0	0	0	0
3	0.48	0.36	0.66	0
4	0	0	0	0.65
5	0	0	0	0
6	0	0	0	0
7	0	0	0	0
8	0.06	0.05	0	0
9	1.86	0.83	1.94	0.05
10	0.08	0.02	0.83	1.20
11	0	0	0	0.10
12	0.28	0.16	0	0
13	0.45	0.02	0	0
14	0	0	T	0.04
15	0	0	0	0
16	0.01	T	0.10	0
17	0	0.02	0	0.05
18	0	0	0	0
19	0.05	0.42	0.15	0
20	T	0.43	0	1.25
21	0	0.55	0	0
22	0.84	T	0.18	0
23	0	0	0.42	0.25
24	0	0	0	0
25	0	0	0	0
16	0	0	0	0
27	0	0	0	0
28	0	0	0	0
29	0	0	0	0
30	T	0.12	0.07	0
31	0	0	0	T

T Designates trace

* Records from U. S. Department of Commerce

Table 9 . DAILY RAINFALL DATA* (WATER EQUIVALENT IN INCHES) METROPOLITAN AREA OF CINCINNATI, OHIO

SEPTEMBER 1970

Date	Abbe Observatory	Greater Cincinnati Airport	Cherry Grove	West Fork Dam
1	0	0	0	0
2	0	T	0	0
3	0.23	0.30	0.37	0
4	0.09	0.14	0.11	0.25
5	0.05	0.01	0	T
6	0	T	0	0
7	0.01	0.11	T	0
8	0.55	0.54	T	0
9	0	0	0	0.70
10	0	0	0	0
11	0	0	0	0
12	0	0	0	0
13	0.55	0.05	0	0
14	0.02	0.02	0.22	1.15
15	0	0	T	0.10
16	0	0	0	0
17	0	0	0	0
18	0.02	0.18	0.42	0
19	0	0	0	0.17
20	0	0	0	0
21	T	0.09	0.04	0
22	0.49	0.42	0	T
23	0	T	0.08	0.25
24	0.30	0.47	0	0
25	0.65	0.72	1.53	0.05
26	0.51	0.74	0.20	0.90
27	0.08	0.08	0.37	0.60
28	0	0	0	0
29	0	0	0	0
30	0	0	0	0

T Designates trace

* Records from U. S. Department of Commerce

Table 9 . DAILY RAINFALL DATA* (WATER EQUIVALENT IN INCHES) METROPOLITAN AREA OF CINCINNATI, OHIO

OCTOBER 1970

Date	Abbe Observatory	Greater Cincinnati Airport	Cherry Grove	West Fork Dam
1	0	0	0	NO DATA AVAILABLE
2	0	0	0	
3	0	0	0	
4	0	0	0	
5	0	0	0	
6	0	0	0	
7	0	0	0	
8	0.08	0.04	T	
9	0.34	0.24	T	
10	0.04	0.06	0.34	
11	0	0	0	
12	0.29	0.27	0.14	
13	0.16	0.21	0.80	
14	0.44	0.48	0.52	
15	0	0.01	0.03	
16	0	0	0	
17	0	0	0	
18	0	0	0	
19	0	0	0	
20	0.89	0.96	0.94	
21	T	T	0.27	
22	0	0	0	
23	0	0	0	
24	T	T	0	
25	0	0	0	
26	0	0	T	
27	0	T	0	
28	0.03	0.02	0.04	
29	0.27	0.15	0.24	
30	T	T	0.29	
31	0	0	0	

T Designates trace

* Records from U. S. Department of Commerce

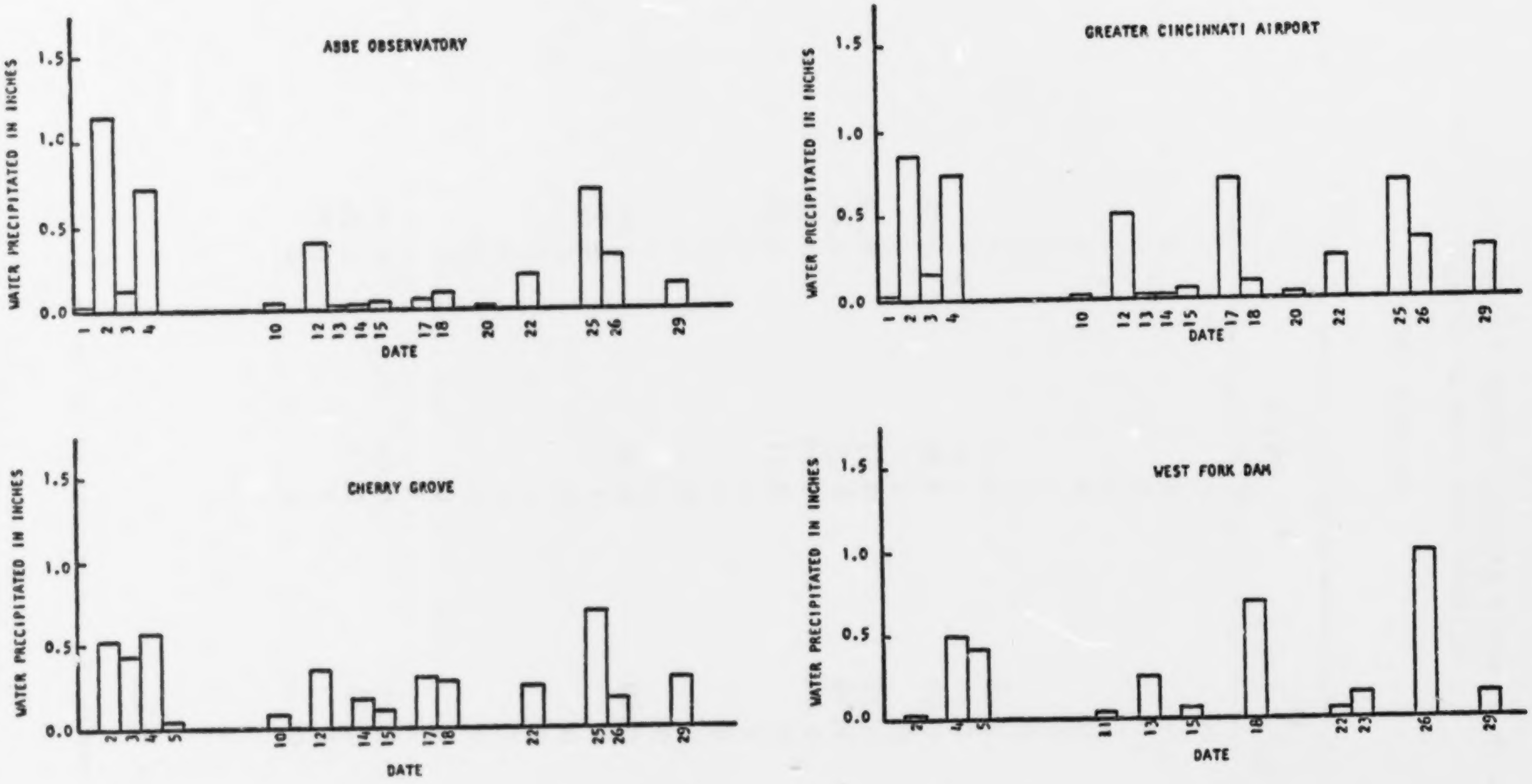


Figure 58. DAILY RAINFALL DATA. MARCH 1970
(Records from U. S. Department of Commerce)
METROPOLITAN AREA OF CINCINNATI, OHIO

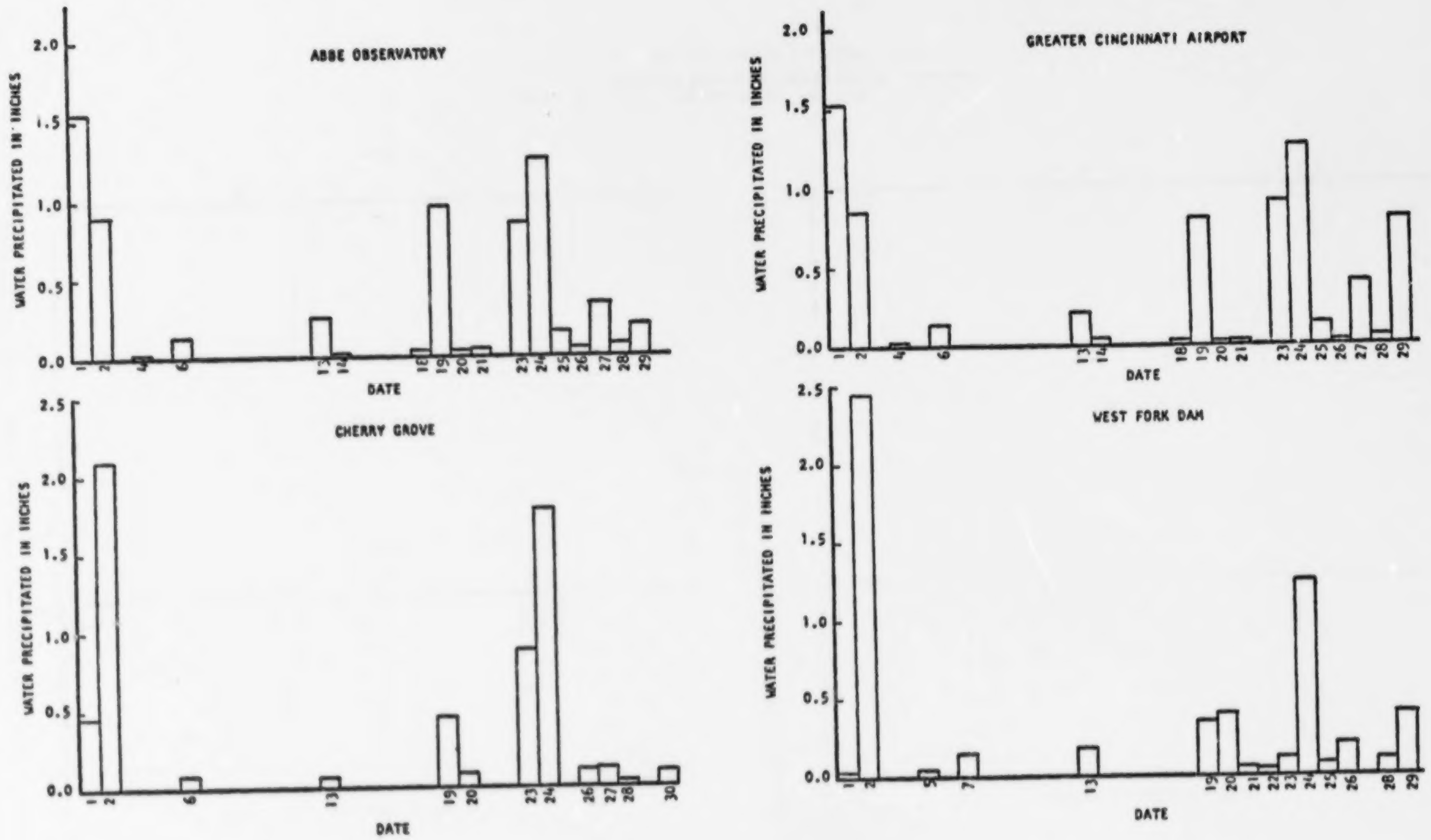


Figure 58. DAILY RAINFALL DATA, APRIL 1970
(Records from U. S. Department of Commerce)
METROPOLITAN AREA OF CINCINNATI, OHIO

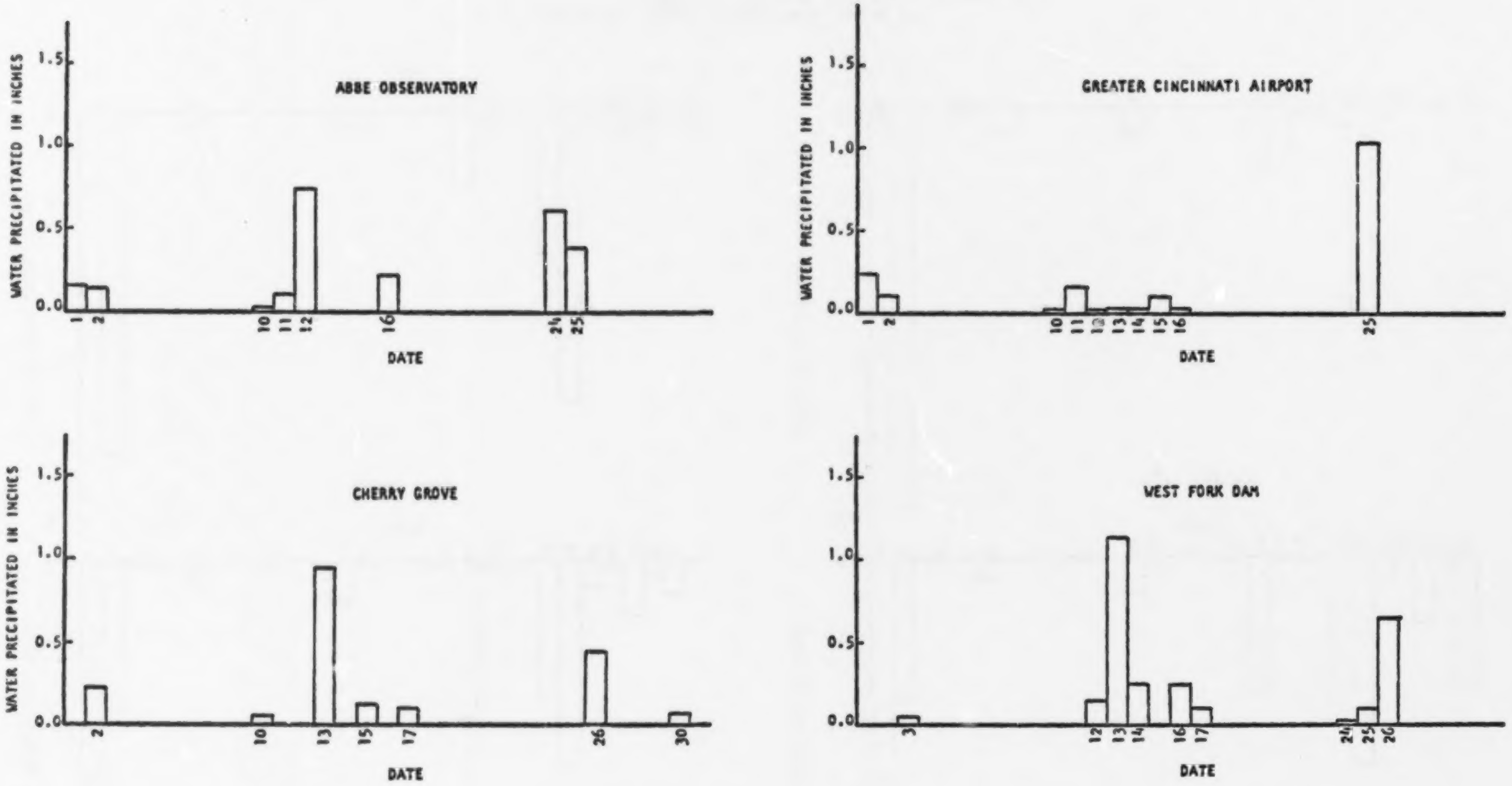


Figure 58. DAILY RAINFALL DATA. MAY 1970
(Records from U. S. Department of Commerce)
METROPOLITAN AREA OF CINCINNATI, OHIO

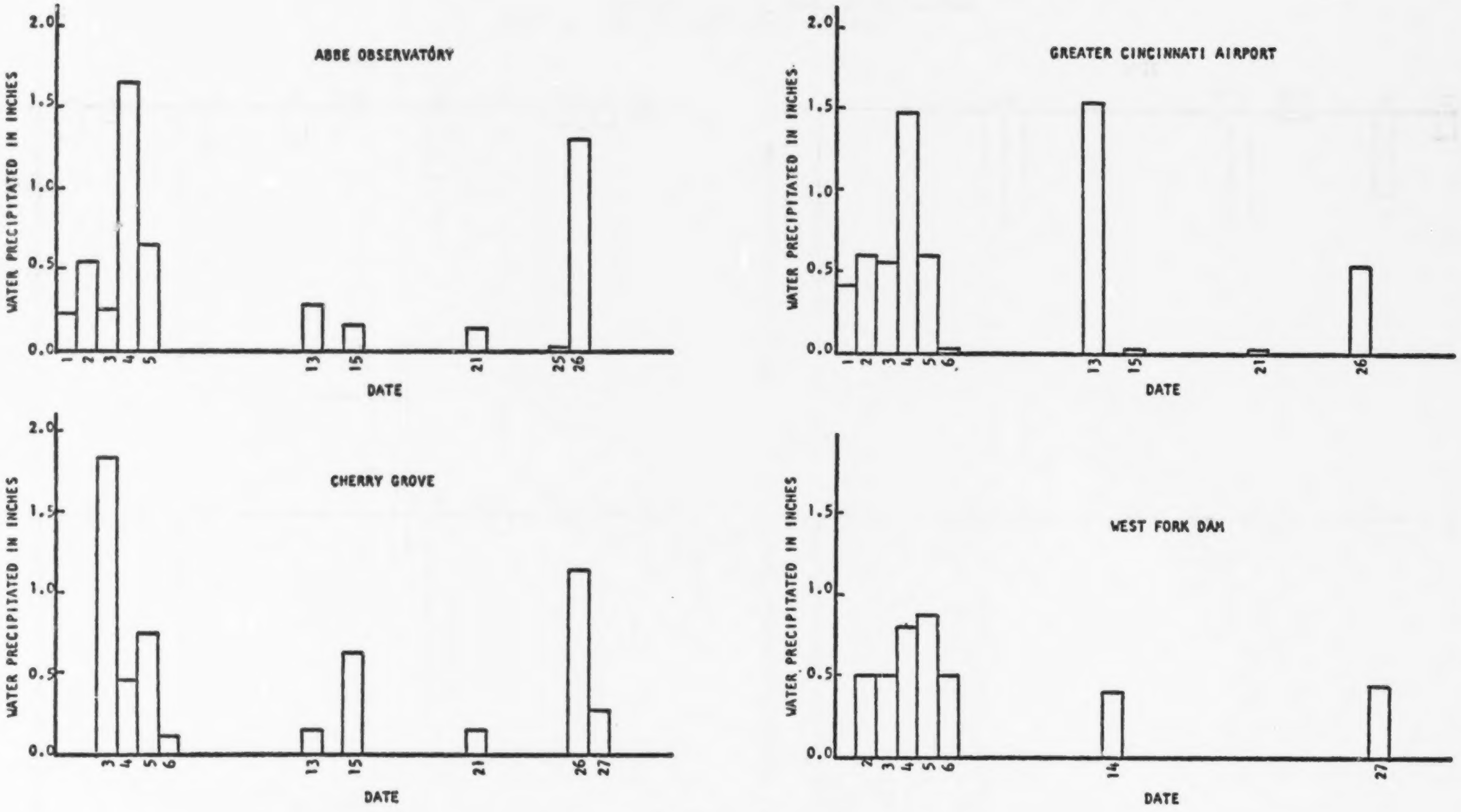


Figure 58 DAILY RAINFALL DATA, JUNE 1970
(Records from U. S. Department of Commerce)

METROPOLITAN AREA OF CINCINNATI, OHIO

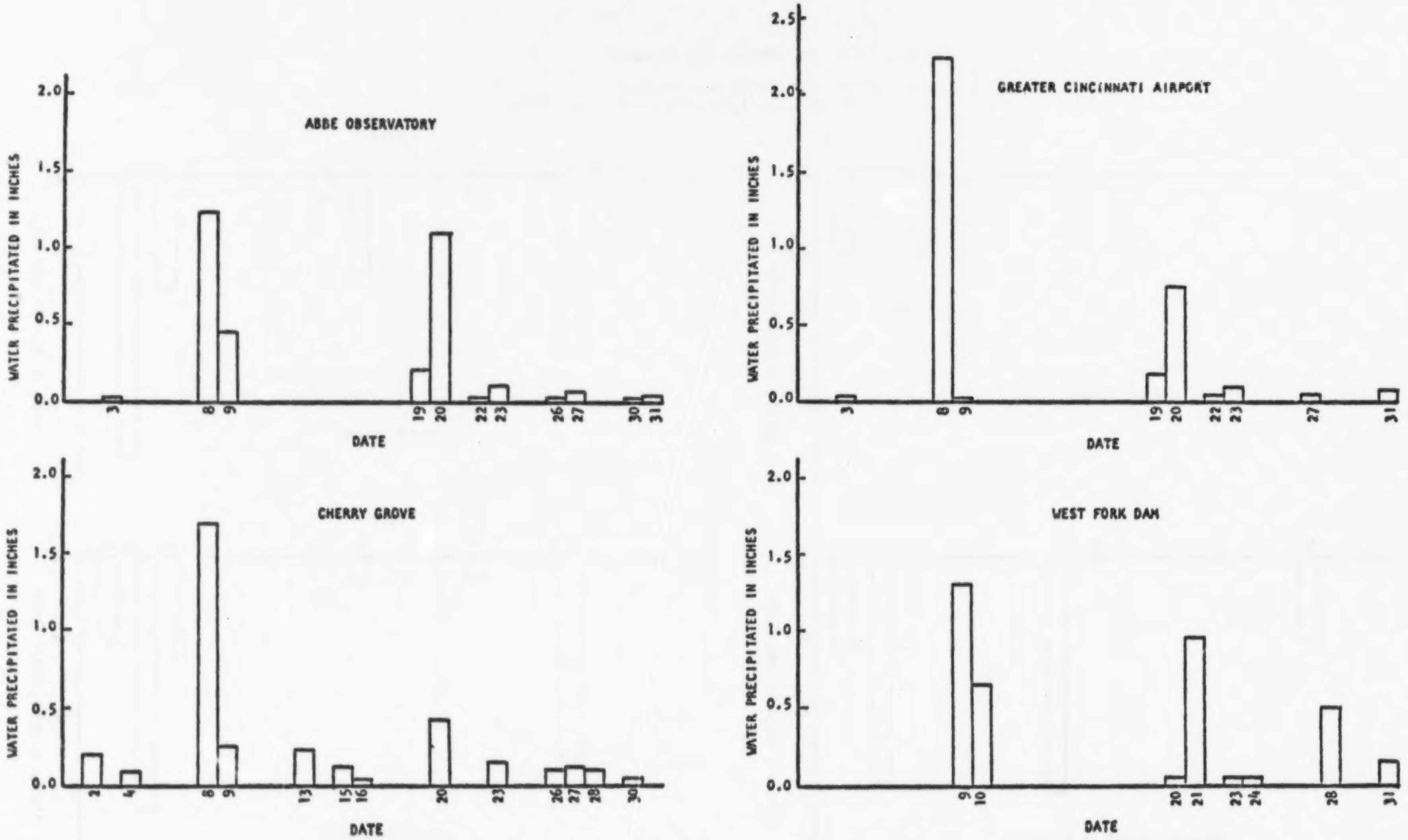


Figure 58 . DAILY RAINFALL DATA, JULY 1970
(Records from U. S. Department of Commerce)
METROPOLITAN AREA OF CINCINNATI, OHIO

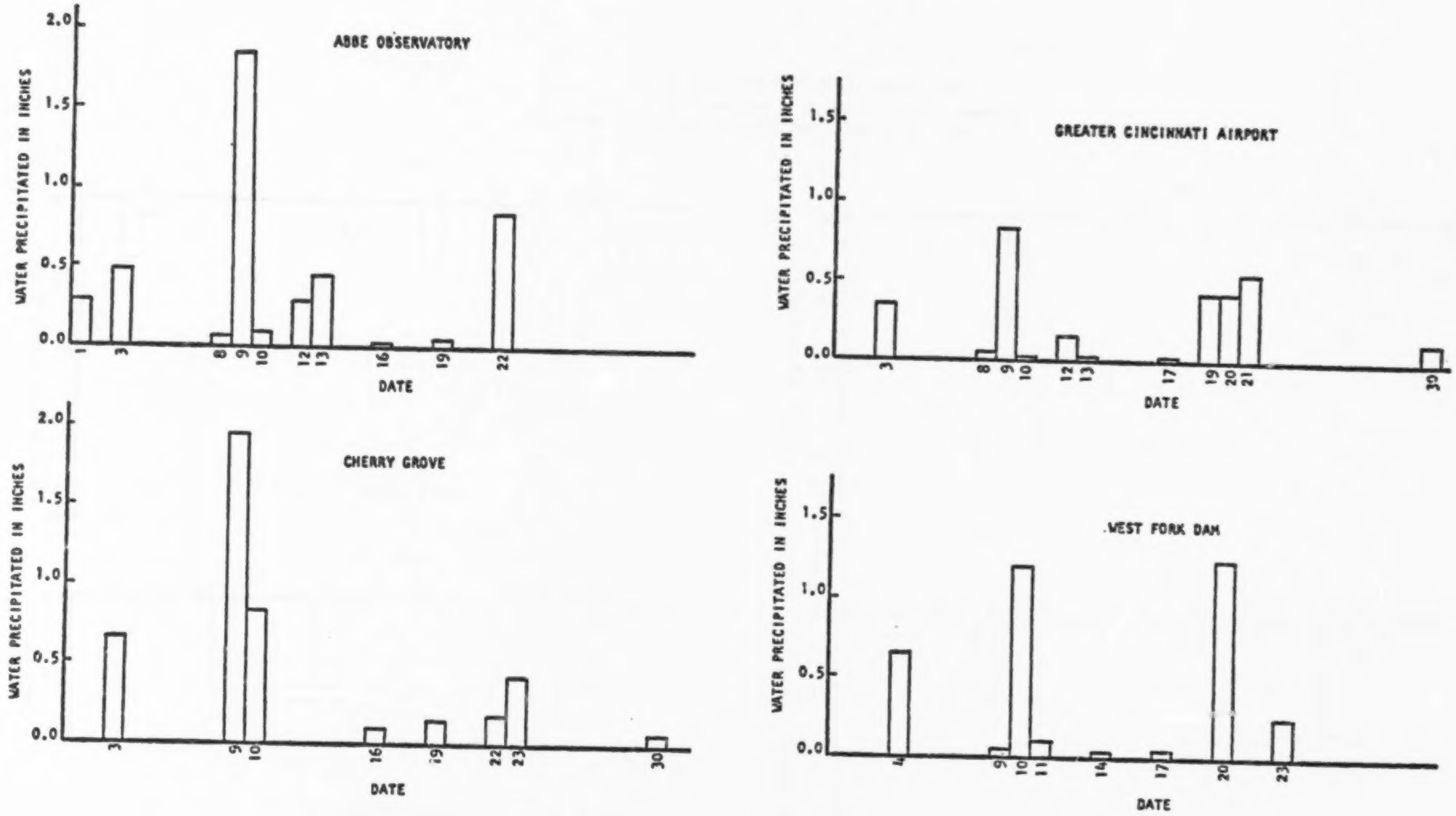


Figure 58. DAILY RAINFALL DATA. AUGUST 1970
(Records from U. S. Department of Commerce)
METROPOLITAN AREA OF CINCINNATI, OHIO

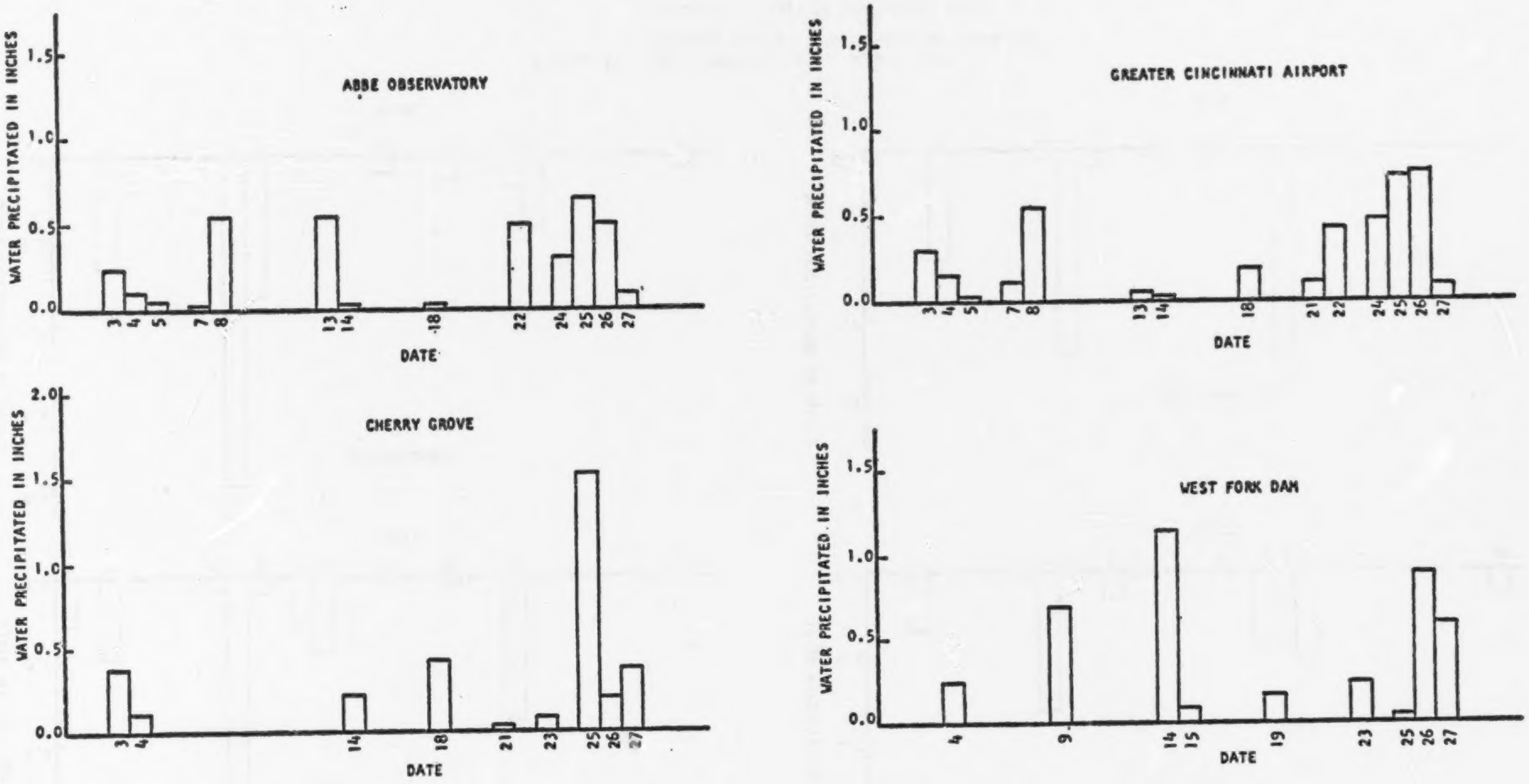


Figure 58. DAILY RAINFALL DATA, SEPTEMBER 1970
(Records from U. S. Department of Commerce)
METROPOLITAN AREA OF CINCINNATI, OHIO

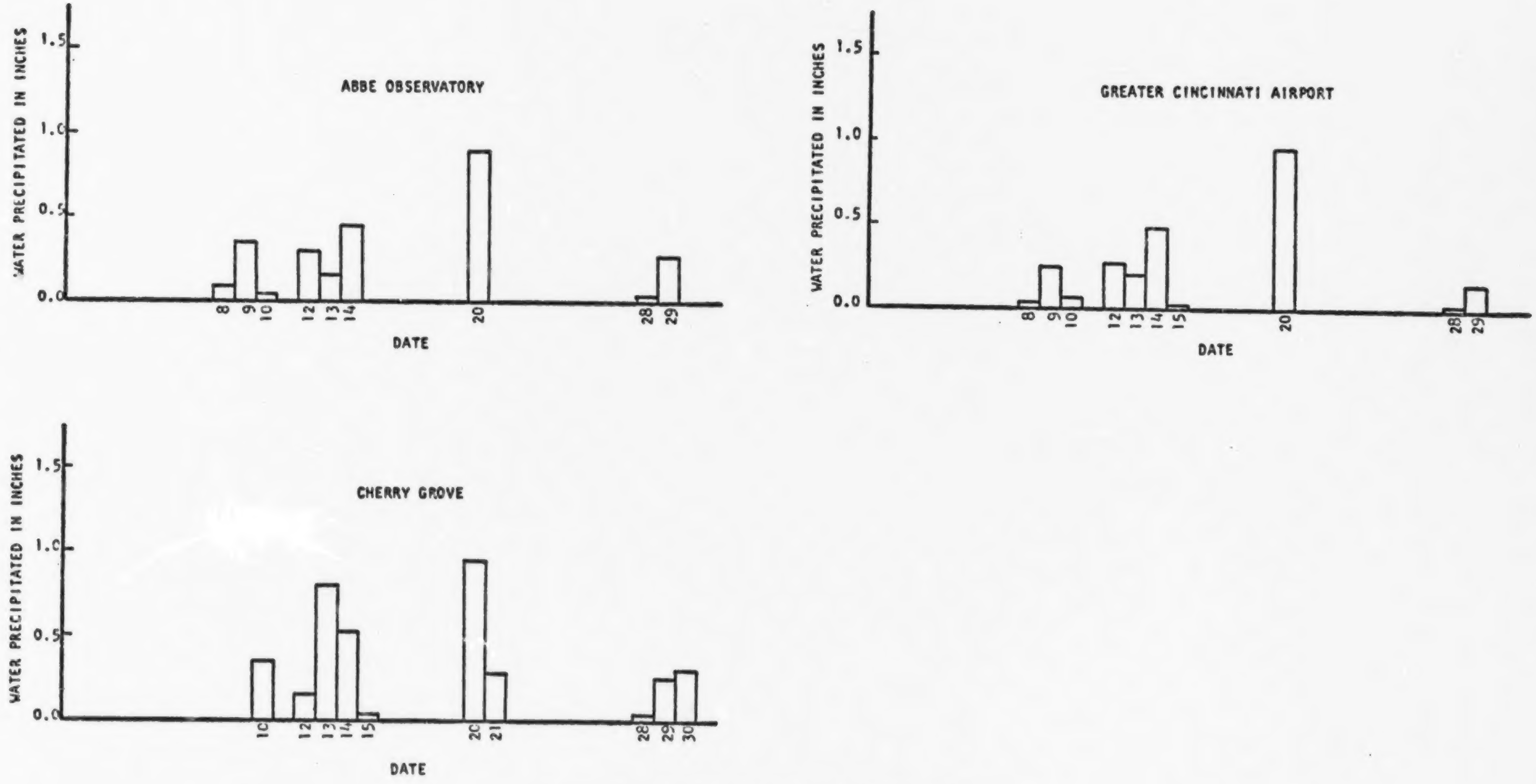


Figure 58. DAILY RAINFALL DATA, OCTOBER 1970
(Records from U. S. Department of Commerce)
METROPOLITAN AREA OF CINCINNATI, OHIO

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2. Dry Weather Flow Data

In addition to the five storm flow monitoring stations which are also able to sample and measure the dry weather flow automatically at any time, six dry weather flow stations were set up temporarily during the summer of 1970 and located as shown in Figures 59 and 60.

Flow quantity and quality data obtained from these stations are presented on Table 10. At the end of this table, the detailed location of each sampling point is given.

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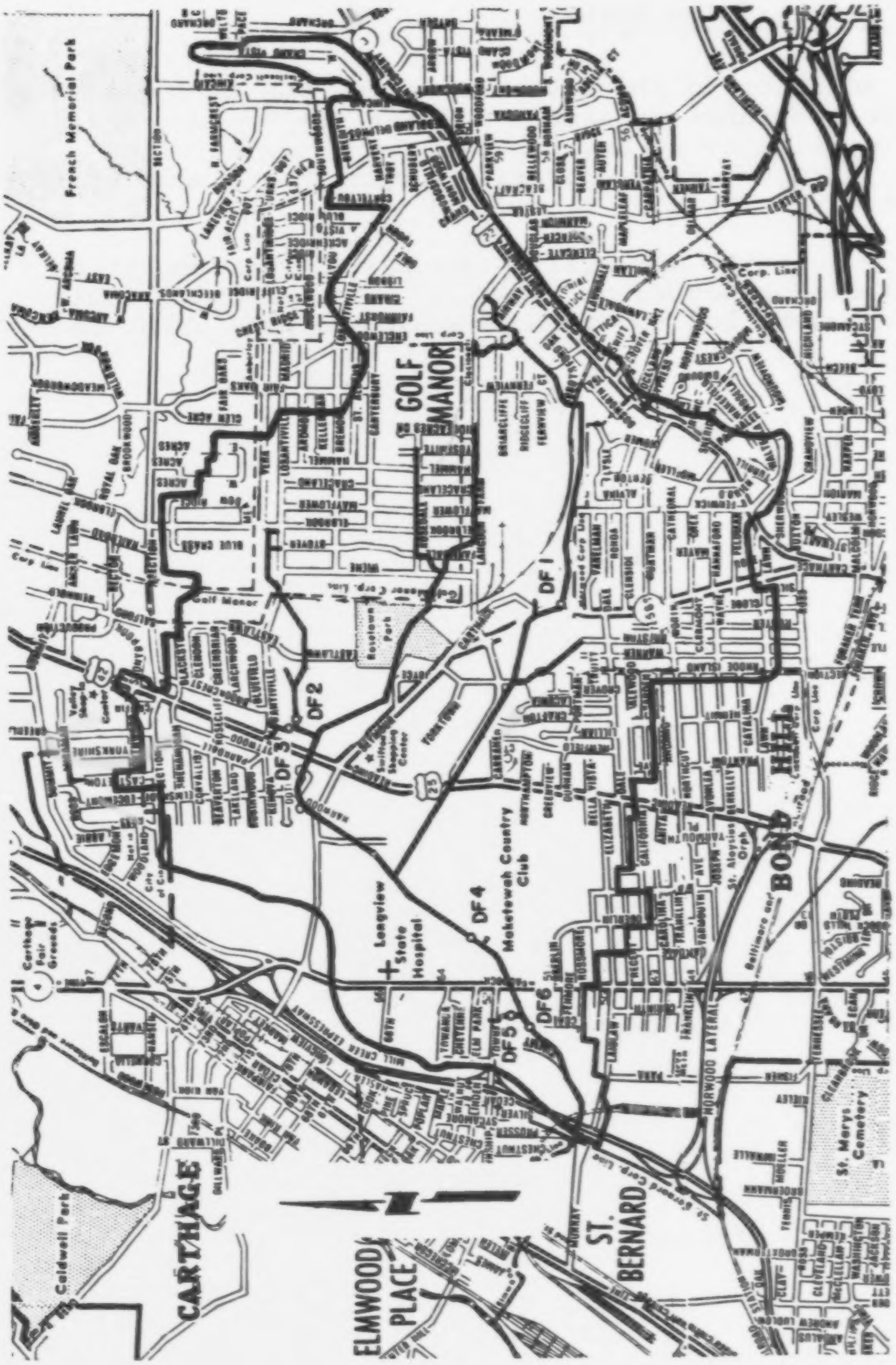
GOLF MANOR
CARTHAGE
ELMWOOD PLACE
ST. BERNARD
BOND HILL
DF 1
DF 2
DF 3
DF 4
DF 5
DF 6
Longview State Hospital
Maketewah Country Club
French Memorial Park
Caldwell Park
Carthage Fair Grounds
Roselawn Park
St. Marys Cemetery

Figure 59. BLOODY RUN SEWER WATERSHED. DRY WEATHER FLOW SAMPLING POINTS.

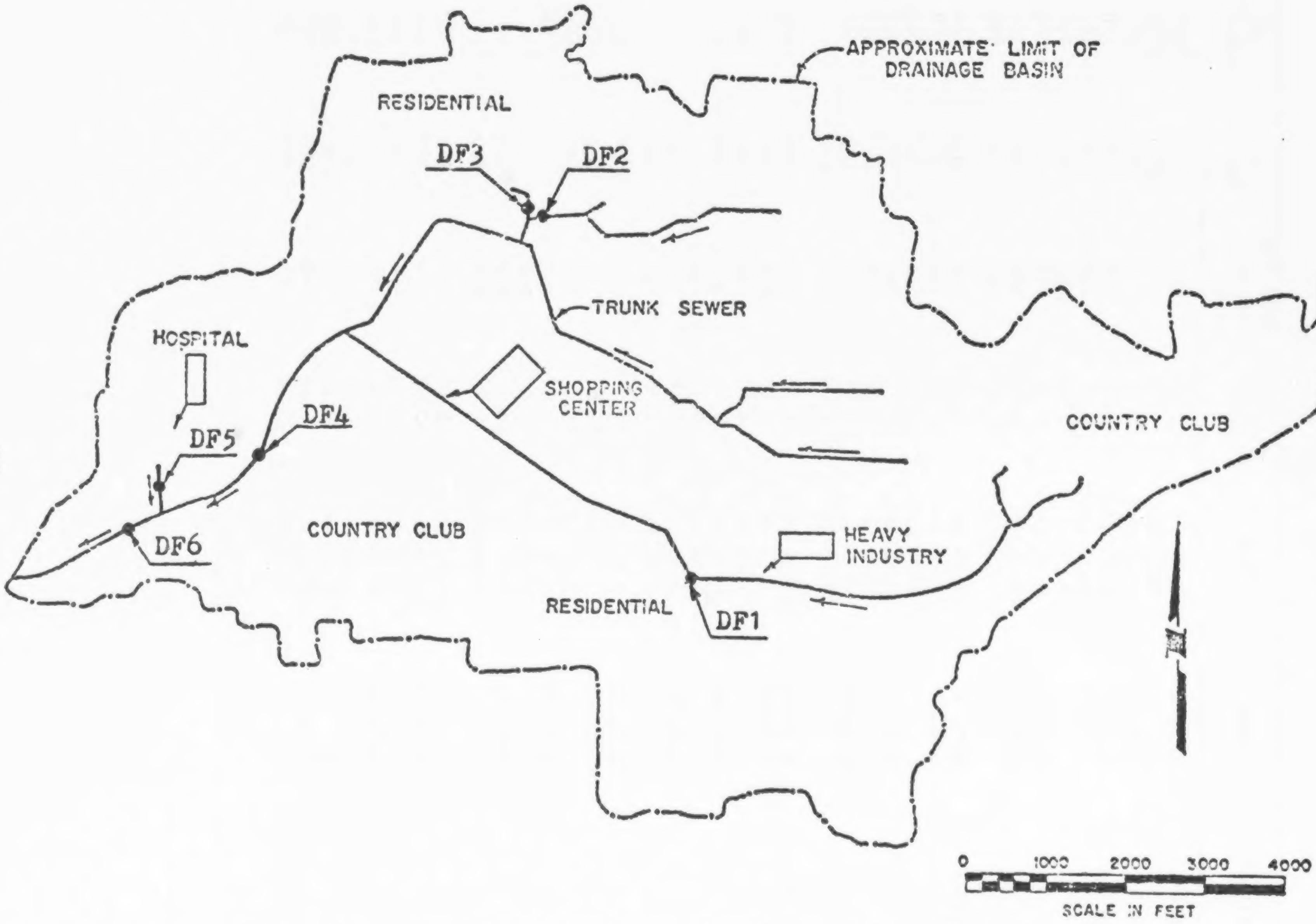
APPROXIMATE LIMIT OF DRAINAGE BASIN
RESIDENTIAL
DF3
DF2
TRUNK SEWER
HOSPITAL
SHOPPING CENTER
COUNTRY CLUB
DF4
DF5
DF6
COUNTRY CLUB
HEAVY INDUSTRY
RESIDENTIAL
DF1
0
1000
2000
3000
4000
SCALE IN FEET

Figure 60. DRY WEATHER SAMPLING POINTS AND MAJOR LAND USE CLASSIFICATIONS

TABLE 10 - DRY WEATHER FLOW
QUANTITATIVE AND QUALITATIVE DATA

Location*	Date	Time	Discharge (cfs)	Suspended Solids (mg/l)	COD (mg/l)	BOD (mg/l)
DF1 (78")	7/10/70	10:30 am	1.26	472	3229	1300
		1:00 pm	1.10	388	9828	5600
	7/12/70	11:00 am	1.15	112	291	240
	7/13/70	12:00 noon	1.24	178	242	210
	7/14/70	9:00 am	2.51	118	1587	980
		2:05 pm	1.53	234	400	360
	7/15/70	9:15 am	1.50	102	2257	2000
	7/16/70	10:30 am	2.20	176	1743	1530
		1:00 pm	1.13	162	920	800
DF2 (78")	6/29/70	11:30 am	1.05	290	181	160
	6/30/70	9:15 am	1.18	224	381	390
		1:45 pm	1.10	152	398	320
	7/01/70	9:00 am	0.85	394	543	420
		1:40 pm	0.58	198	495	410
	7/02/70	9:30 am	0.81	162	582	--
		2:00 pm	0.99	138	444	--
DF3 (54")	6/29/70	11:30 am	0.52	176	398	385
	6/30/70	9:15 am	0.55	470	398	--
		1:45 pm	0.65	284	362	380
	7/01/70	9:00 am	0.49	308	343	350
		1:40 pm	0.53	126	--	--
	7/02/70	9:30 am	0.56	118	160	--
		2:00 pm	0.50	130	269	--
DF4 (10' x 15')	6/29/70	2:45 pm	13.7	184	1629	450
	6/30/70	1:15 pm	15.6	220	1882	700
		2:40 pm	18.5	216	1520	600
	7/01/70	9:45 am	13.8	303	1158	720
		2:30 pm	15.6	228	757	--
	7/02/70	10:20 am	14.9	482	2519	--
		3:00 pm	16.7	220	1340	--
DF5 (54")	7/10/70	10:50 am	0.50	256	364	160
	7/13/70	11:30 am	0.50	274	261	150
		1:15 pm	0.50	150	546	290
	7/14/70	9:30 am	0.50	142	463	210
		1:30 pm	0.50	138	364	440
	7/15/70	9:40 am	0.50	198	3495	260
	7/16/70	10:40 am	0.50	142	562	495
		1:20 pm	0.50	146	350	330

Table 10 (continued)

Location*	Date	Time	Discharge (cfs)	Suspended Solids (mg/l)	COD (mg/l)	BOD (mg/l)
DF6 (10' x 15')	7/10/70	11:40 am	14.77	385	2063	1000
		1:30 am	12.13	346	6552	4100
	7/12/70	11:10 am	11.82	174	650	590
	7/13/70	11:30 am	15.01	252	400	180
		1:00 pm	14.50	282	386	220
	7/14/70	9:40 am	15.02	238	815	610
		1:30 pm	12.38	288	466	380
	7/15/70	9:45 am	14.8	168	728	670
	7/16/70	10:20 am	15.12	156	673	596
		1:20 pm	13.85	230	392	374

*Locations where the dry weather flow measurements have been taken

DF1	At Seymour Avenue about 300 yards southeast from Cincinnati Gardens and about 300 yards downstream from Hilton Davis Chemical Company.
DF2 DF3	At the location of Monitoring Station "BANK"
DF4	At the golf course of Maketawah Country Club.
DF5	About 75 yards from the corner of Coad and Lois Drive.
DF6	About 100 yards from the corner of Coad and Lois Drive.

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3. Rainfall and Runoff Data from the Bloody Run Sewer Watershed

In the following pages, the data collected from the five monitoring locations of the Bloody Run Sewer Watershed are presented.

The laboratory analyses of the samples taken at the monitoring stations have been carried out according to the standard test procedures in "Standard Methods for the Examination of Water and Waste Water", 12th Edition, 1965.

A list of the tests performed and their page numbers in the reference are given below:

(1) Total Solids (TS)	p. 244
(2) Suspend Solids (SS)	p. 246
(3) Volatile Suspended Solids (VSS)	p. 247
(4) Biological Oxygen Demand (BOD)	p. 415
(5) Chemical Oxygen Demand (COD)	p. 510
(6) Chlorides (Cl)	p. 370
(7) pH	p. 422

The data tabulation has been set up in a comprehensive pattern. For every rainfall, a plot of the rain hyetographs comes first and five tables follow with the field data corresponding to the five monitoring locations. Each one of these five tables is accompanied by a plot of the data versus time for the convenience of the reader.

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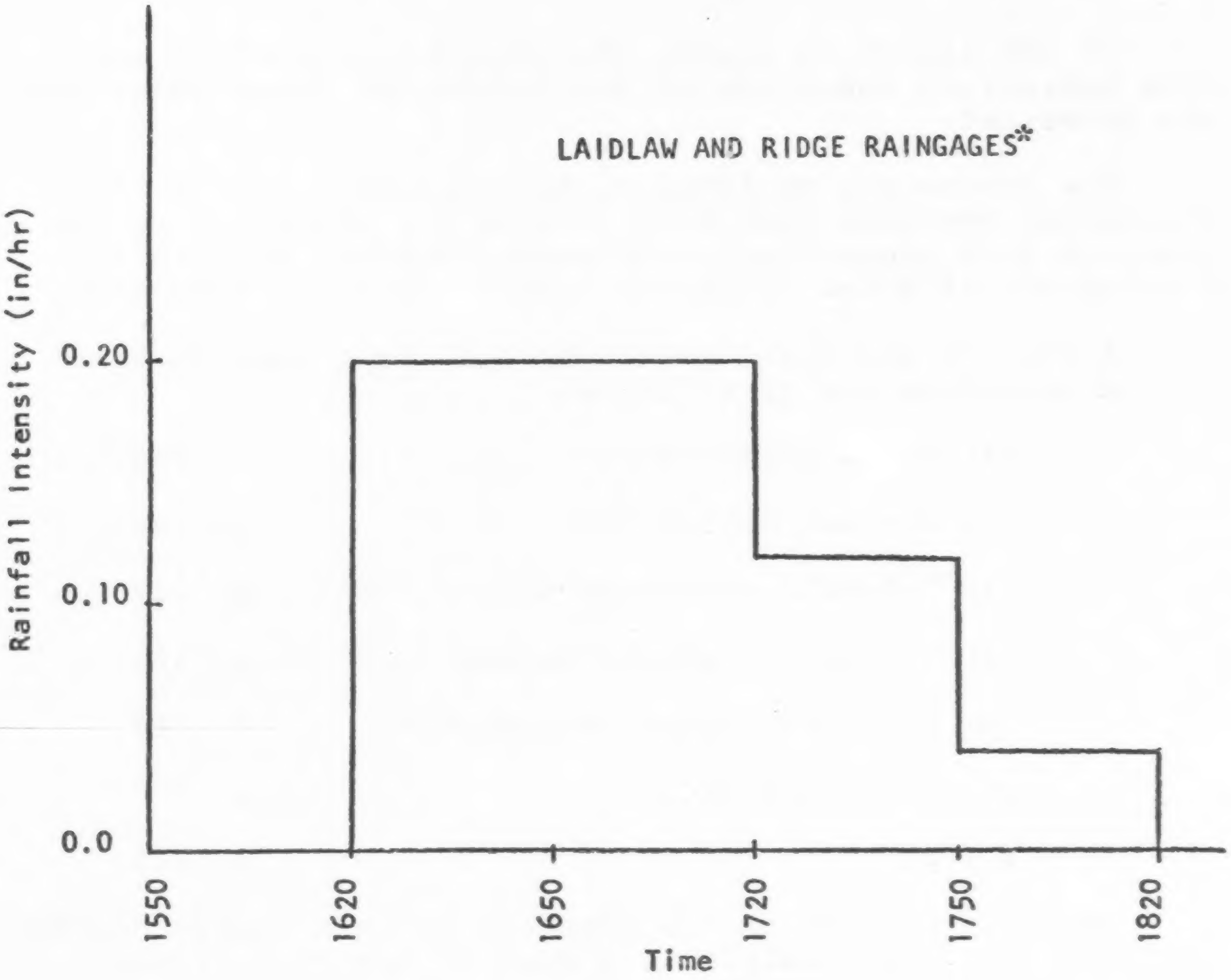


Figure 61 . RAIN OF APRIL 1, 1970
HYETOGRAPH

*Until August 1971 these two raingages had a 7 day rotation period; therefore, no difference in readings from the charts could be detected.

Table 11 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: April 1, 1970; MONITORING STATION: Bank - 54" Sewer

Time	Intensity of Rain (in/hr) Laidlaw Gage	Intensity of Rain (in/hr) Ridge Gage	Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	Volatile Suspended Solids mg/l	BOD	COD
1550	0.00	0.00						
1620	0.20	0.20						
1650	0.20	0.20						
1655			7.5	1400	1125	300	200	365
1702:30			10.0	1450	1200	175	140	280
1710			12 5	1700	1500	150	120	240
1717:30			14.0	1775	1550	150	115	155
1720	0.12	0.12						
1725			14.0	1700	1450	200	115	160
1732:30			12.0	1650	1300	500	125	125
1740			11.0	1425	1125	500	125	210
1747:30			8.5	1150	925	425	140	215
1750	0.04	0.04						
1755			6.5	1325	800	300	125	250
1802:30			3.0	1200	550	175	200	300
1810			3.5	1025	650	350	215	325
1817:30			2.5	1000	500	225	220	390
1820	0.00	0.00						
1825			1.5	425	420	170		

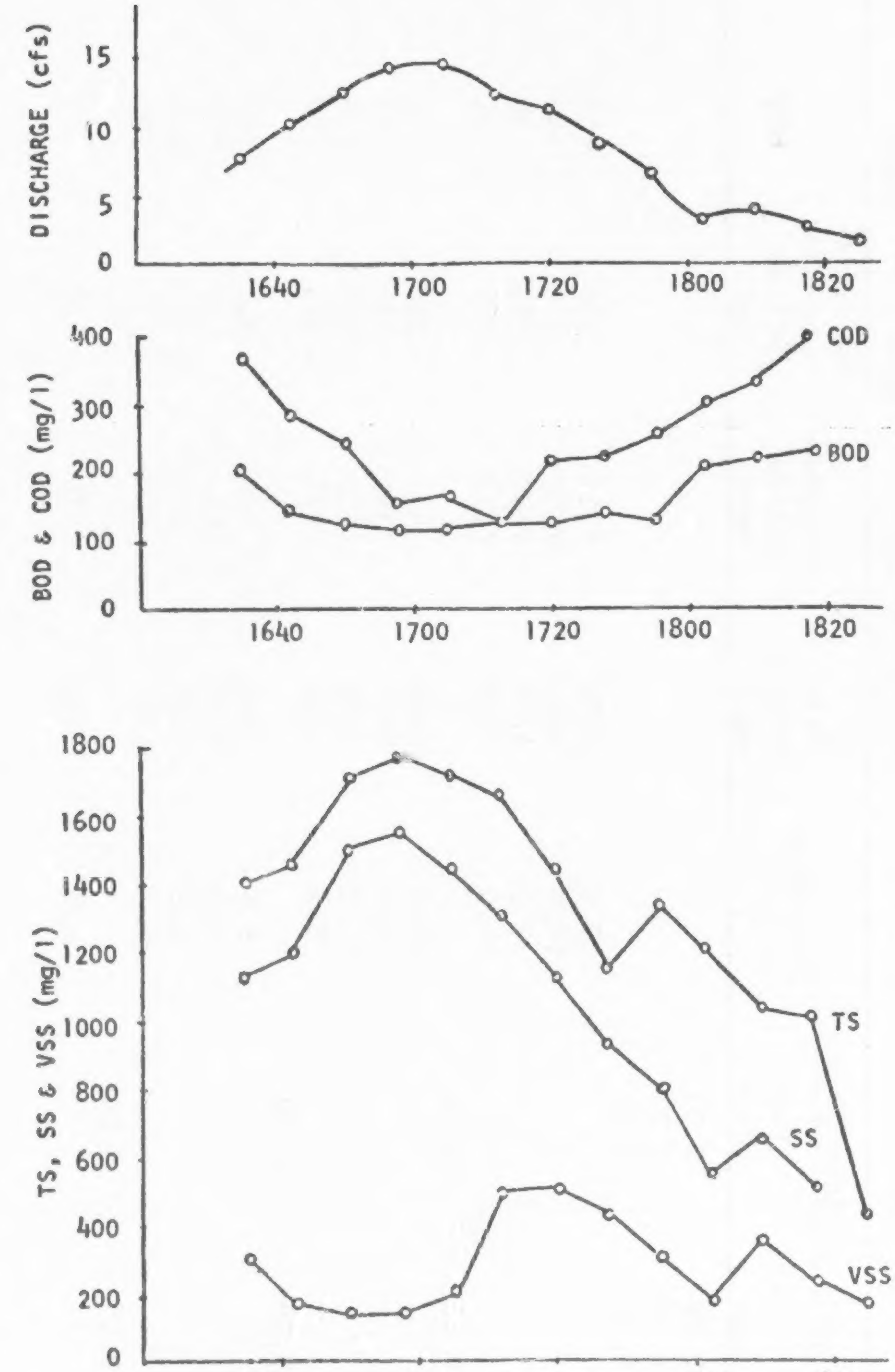


Figure 62. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: APRIL 1, 1970
MONITORING STATION: BANK - 54" SEWER

Table 12 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: April 1, 1970 MONITORING STATION: Bank - 78'' Sewer

Time	Intensity of Rain (in/hr)		Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	Volatile Suspended Solids mg/l	BOD	COD
	Laidlaw Gage	Ridge Gage						
1550	0.00	0.00						
1620	0.20	0.20						
1650	0.20	0.20						
1705			9.0	1625	950	300	250	450
1712:30			13.25	1800	1050	350	140	250
1720	0.12	0.12	25.5	1925	1325	275	95	125
1727:30			24.5	1625	1400	100	115	130
1735			23.71	1575	1250	225	115	130
1742:30			20.81	1300	1050	125	155	200
1750	0.04	0.04	17.25	1375	1000	175	140	210
1757:30			12.53	1050	800	325	180	250
1805			8.23	950	825	225	230	300
1812:30			5.41	700	500	150	250	350
1820	0.00	0.00	4.63	625	475	250	250	450
1827:30			3.51	625	400	125	270	450
1835			1.60					

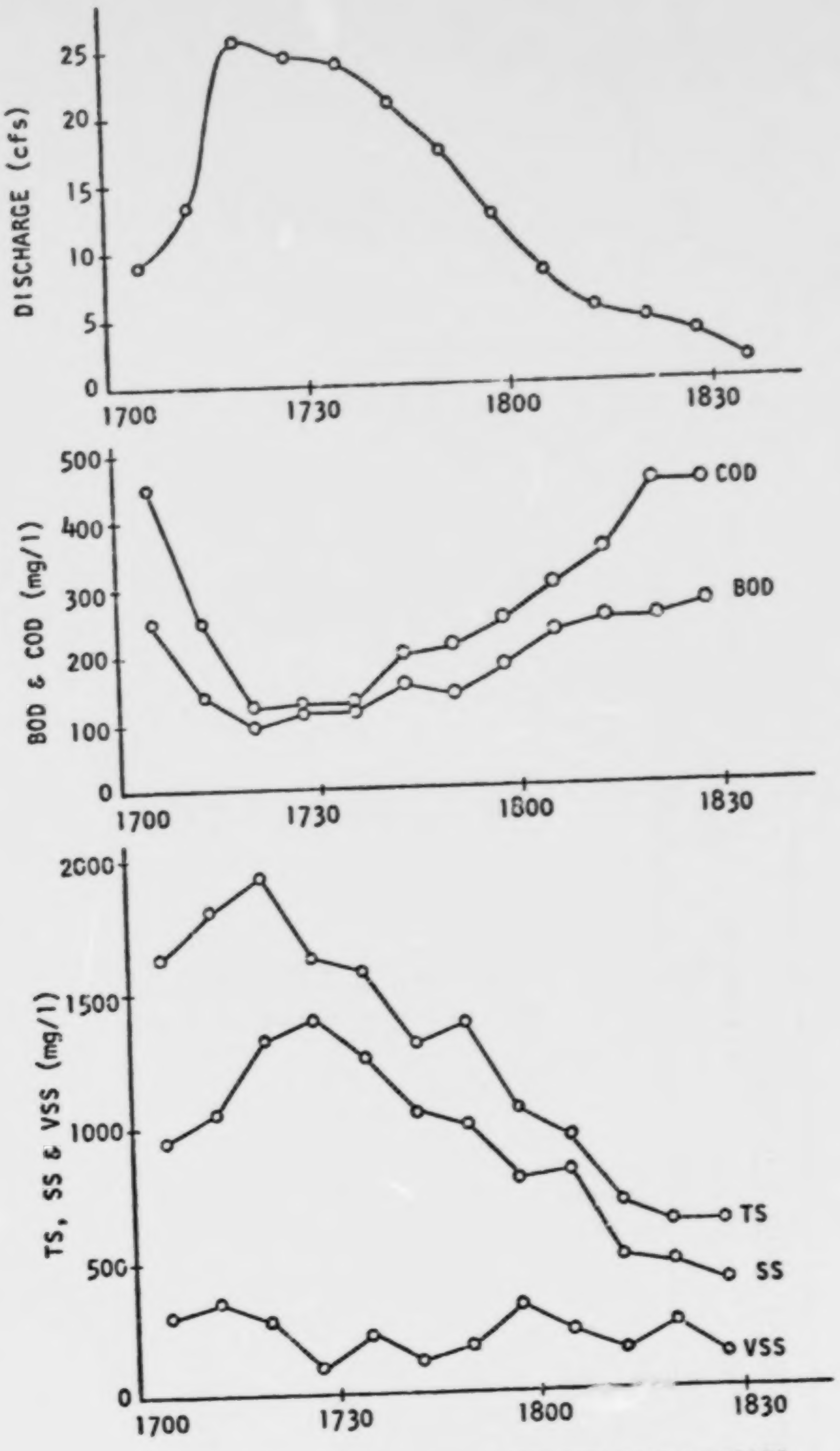


Figure 63. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: APRIL 1, 1970
MONITORING STATION: BANK - 78" SEWER

Table 13 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: April 1, 1970 MONITORING STATION: Outlet - 10' x 15' Sewer

Time	Intensity of Rain (in/hr) Laidlaw Gage	Intensity of Rain (in/hr) Ridge Gage	Discharge in cfs	Total Solids mg/1	Suspended Solids mg/1	Volatile Suspended Solids mg/1	BOD	COD
1550	0.00	0.00						
1620	0.20	0.20						
1650	0.20	0.20						
1720	0.12	0.12	96.0					
1727:30			282.5	2260	1870	610	185	240
1735			311.6	2480	1910	490	100	200
1742:30			301.3	2650	1820	580	94	203
1750	0.04	0.04	292.7	1880	1980	630	210	231
1757:30			256.4	1470	1050	280	254	269
1805			184.3	1280	920	270	273	294
1812:30			163.2	1020	840	260	298	315
1820	0.00	0.00	141.7	940	770	250	337	347
1827:30			111.4	910	790	310	368	370
1835			101.3	850	610	150	412	425

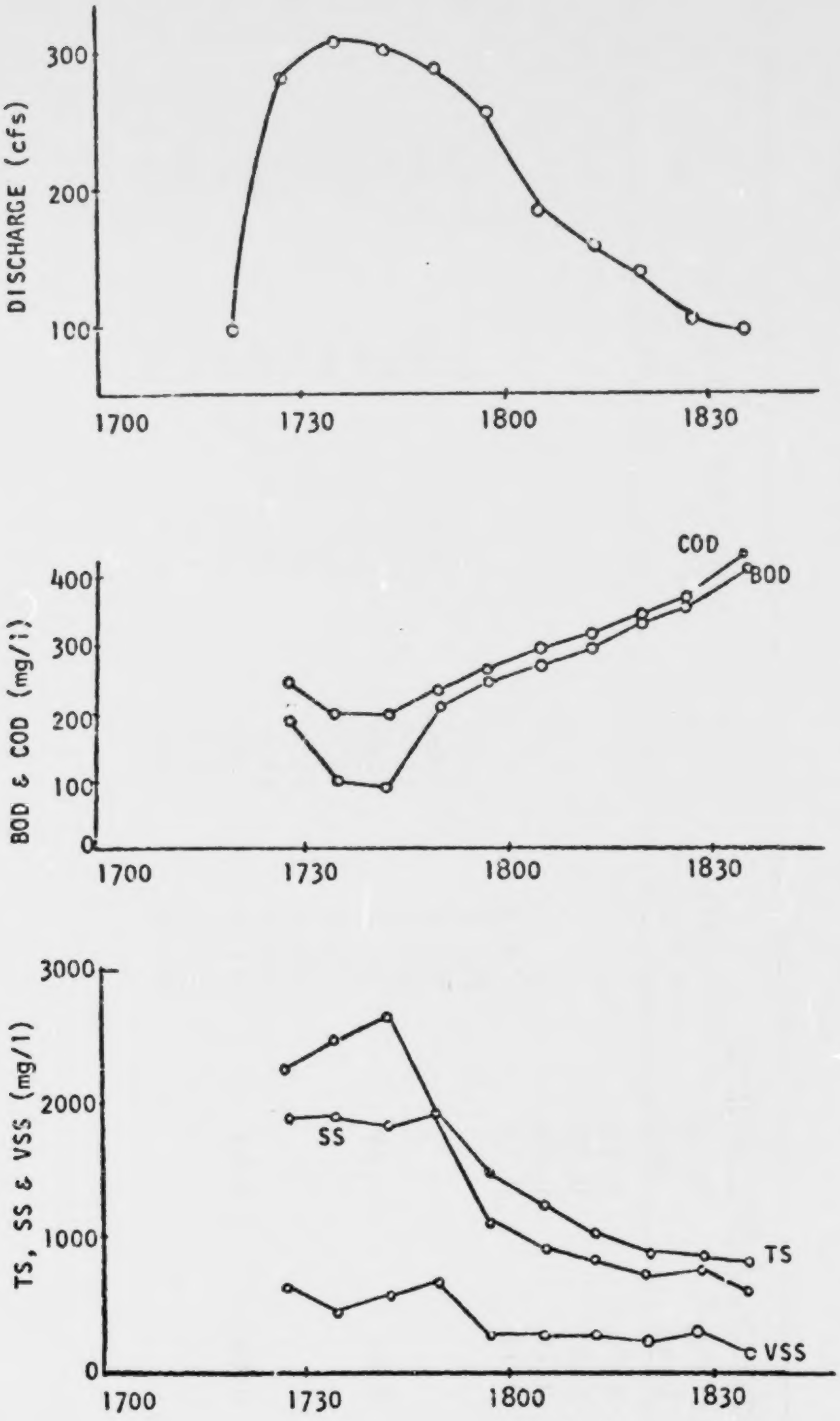


Figure 64. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: APRIL 1, 1970
MONITORING STATION: OUTLET - 10' x 15' SEWER

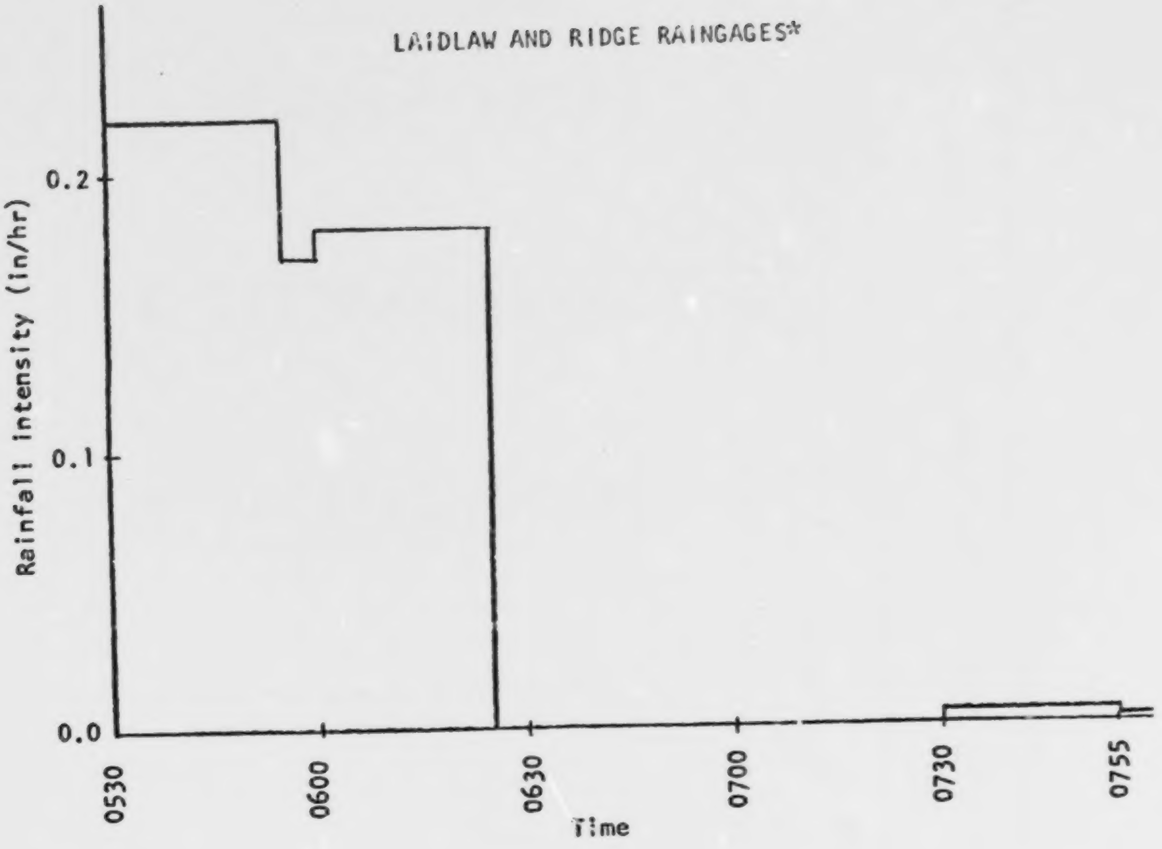


Figure 65. RAIN OF APRIL 24, 1970
HYETOGRAPH

*Until August 1971 these two raingages had a 7 day rotation period; therefore, no difference in readings from the charts could be detected.

Table 14 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: April 24, 1970 MONITORING STATION: Bank - 54'' Sewer

Time	Intensity of Rain (in/hr)		Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	Volatile Suspended Solids mg/l	BOD	COD
	Laidlaw Gage	Ridge Gage						
0500	0.00	0.00						
0530	0.22	0.22						
0540			1.0	1025	915	137	221	422
0547:30			1.5	1065	926	128	201	346
0555	0.18	0.18	2.6	1109	968	156	186	320
0602:30			5.3	1152	1051	199	172	262
0610			9.1	1164	1069	163	142	271
0617:30			11.7	1244	1131	177	-	215
0625	0.00	0.00	11.6	1292	1192	136	121	130
0632:30			10.2	1325	1262	150	92	109
0640			8.7	1295	1206	100	98	122
0647:30			7.8	1262	1152	100	116	129
0655	0.00	0.00	6.5	1192	1106	113	118	-
0702:30			5.3	1161	1061	136	165	192
0710			3.5	1051	926	34	195	265
0717:30			2.8	1011			202	312
0725			1.1	926	898	131	212	378

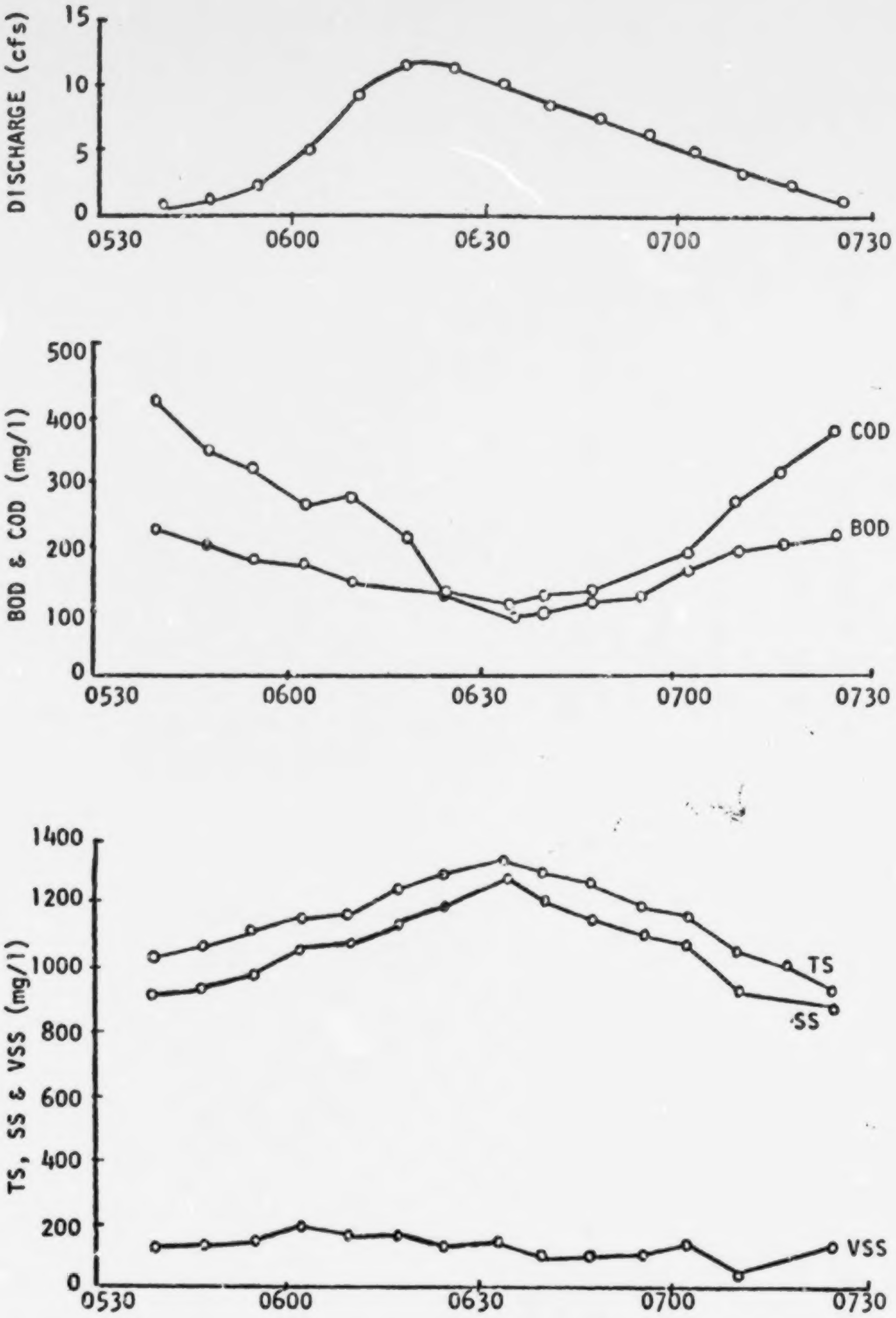


Figure 66. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: APRIL 24, 1970
MONITORING STATION: BANK - 54" SEWER

Table 15 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: April 24, 1970 MONITORING STATION: Bank - 78" Sewer

Time	Intensity of Rain (in/hr)		Discharge in cfs	Total Solids mg/1	Suspended Solids mg/1	Volatile Suspended Solids mg/1	BOD	COD
	Laidlaw Gage	Ridge Gage						
0500	0.00	0.00						
0530	0.22	0.22						
0542			1.3	1124	922	131	211	413
0552	0.17	0.17	15.3	1216	964	165	166	324
0602			19.2	1325	997	184	141	241
0607			23.5	1417	1012	135	111	168
0612			22.3	1478	1215	303	105	114
0622			17.6	1501	1311	146	103	107
0632	0.00	0.00	10.5	1414	1201	150	109	123
0642			16.5	1312	1192	280	106	109
0652			20.9	1308	1068	169	105	106
0702	0.00	0.00	15.8	1211	1022	221	133	169
0712			10.5	1168	906	194	174	271
0722			3.1	1016	892	130	214	392

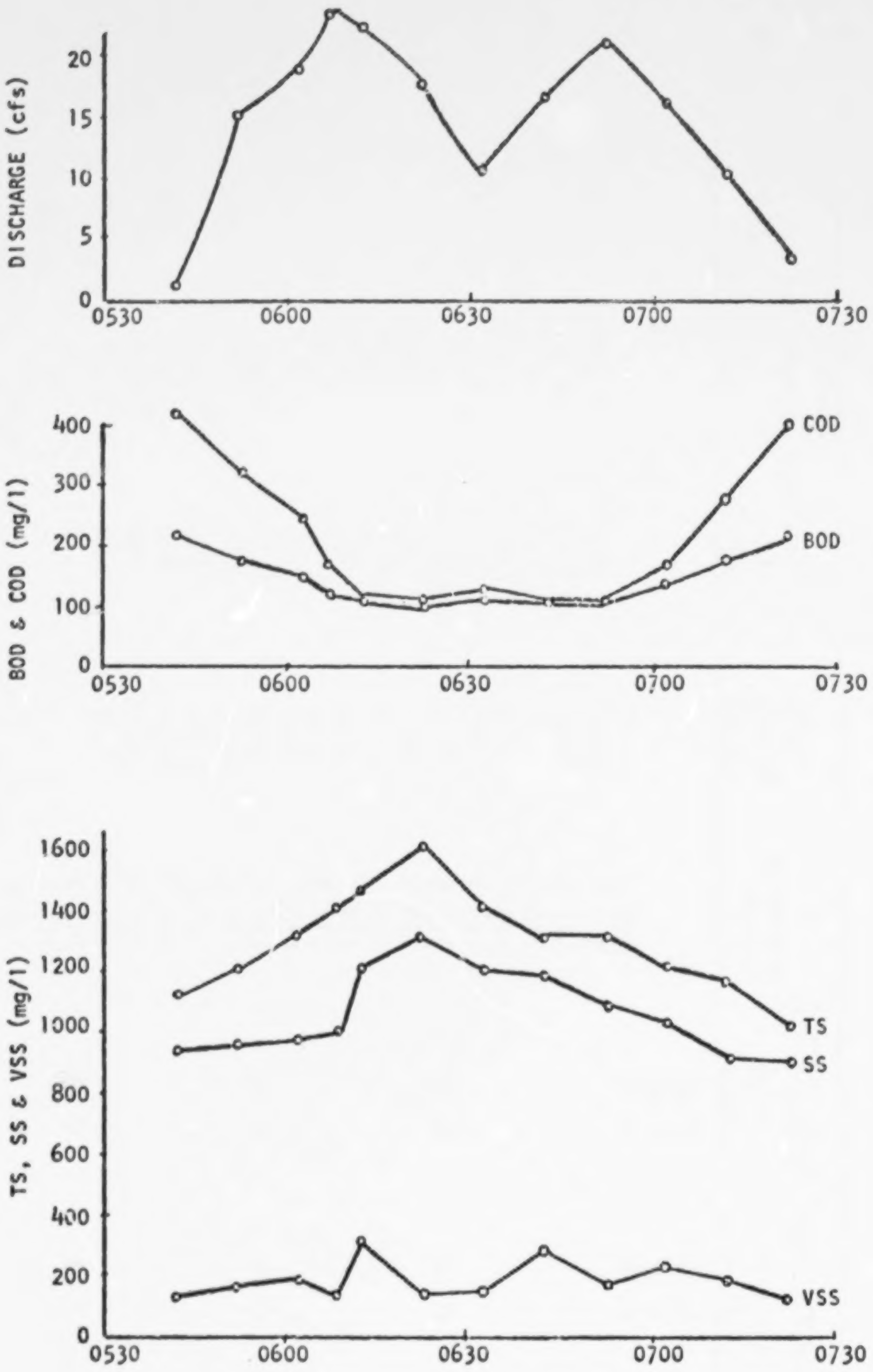


Figure 67. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: APRIL 24, 1970
MONITORING STATION: BANK - 78" SEWER

Table 16 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: April 24, 1970 MONITORING STATION: Outlet - 10' x 15' Sewer

Time	Intensity of Rain (in/hr) Laidlaw Gage	Intensity of Rain (in/hr) Ridge Gage	Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	Volatile Suspended Solids mg/l	BOD	COD
0500	0.00	0.00						
0530	0.22	0.22						
0600	0.18	0.18	96.0					
0605			120.2	2740	1620	1130	150	270
0610			123.3	1990	1830	1110		
0615			126.5	1120	910	100	107	236
0620			128.3	1440	1060	130		
0625			130.1	1480	1240	220	110	225
0630	0.00	0.00	131.6	2060	1680	220		
0635			134.6	2990	1720	130	108	195
0640			138.0	3070	1870	250		
0645			143.1	2210	1450	140	110	285
0650			138.3	2040	1430	150		
0655			136.5	1770	1360	170	115	310
0700	0.00	0.00	134.5	1560	1310	230		
0705			134.3	1240	960	220	110	234
0710			133.2	1160	1020	340		
0715			132.3	1030	840	220	118	270
0720			137.2	940	790	250		
0725			140.3	870	680	190	135	430
0730	0.03	0.03	142.2					
0735			148.1				128	300
0740			134.6					
0745			131.3				117	291
0755	0.01	0.01	117.0				103	285

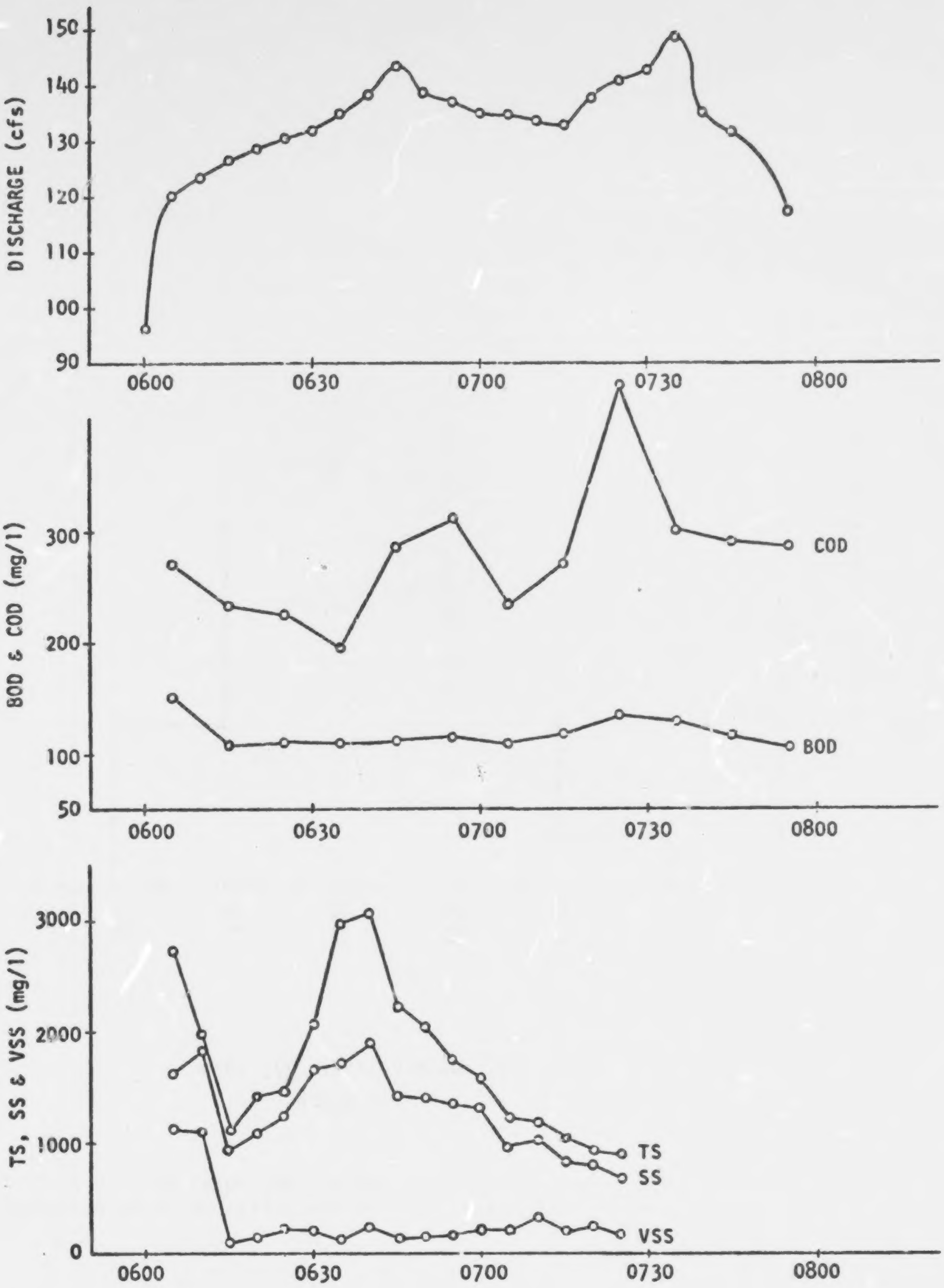


Figure 68. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: APRIL 24, 1970
MONITORING STATION: OUTLET - 10' x 15' SEWER

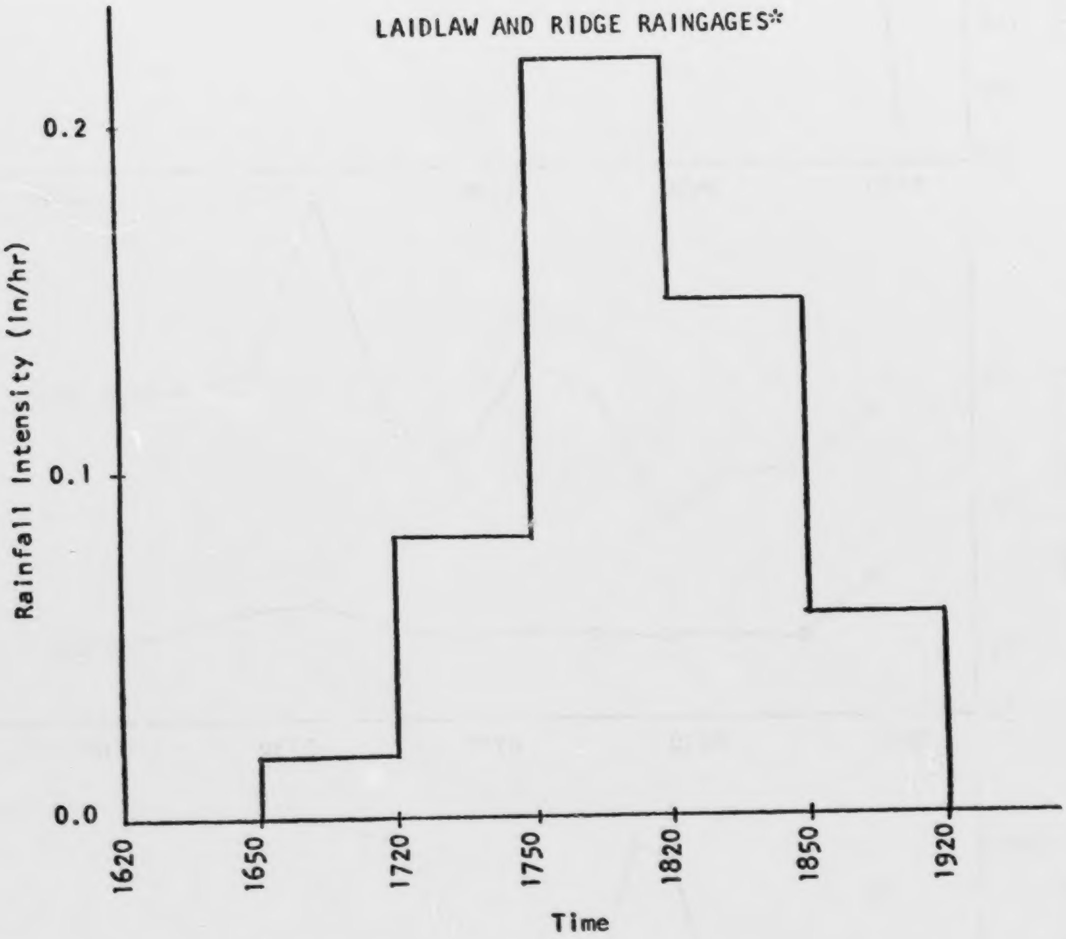


Figure 69. RAIN OF APRIL 27, 1970
HYETOGRAPH

*Until August 1971 these two raingages had a 7 day rotation period; therefore, no difference in readings from the charts could be detected.

Table 17 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: April 27, 1970 MONITORING STATION: Bank - 78" Sewer

Time	Intensity of Rain (in/hr)		Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	Volatile Suspended Solids mg/l	BOD	COD
	Laidlaw Gage	Ridge Gage						
1620	0.00	0.00						
1630			1.25	1251	656	144	265	405
1637:30			3.11	1292	752	101	202	368
1645			6.20	1356	769	77	185	322
1650	0.018	0.018						
1652:30			9.12	1385	812	100	125	256
1700			15.36	1452	865	96	105	212
1707:30			25.10	1513	956	30	92	182
1715			24.36	1501	922	15	94	173
1720	0.081	0.081						
1722:30			20.62	1452	865	13	109	191
1730			17.56	1212	812	-	109	219
1737:30			13.69	--	799	-	-	255
1745			11.21	1109	686	-	165	291
1750	0.218	0.218						
1752:30			15.27	1352	712	-	141	221
1800			11.60	1185	701	45	176	256
1807:30			6.05	1212	686	170	192	315
1815			4.12	1052	615	114	213	345
1820	0.148	0.148						

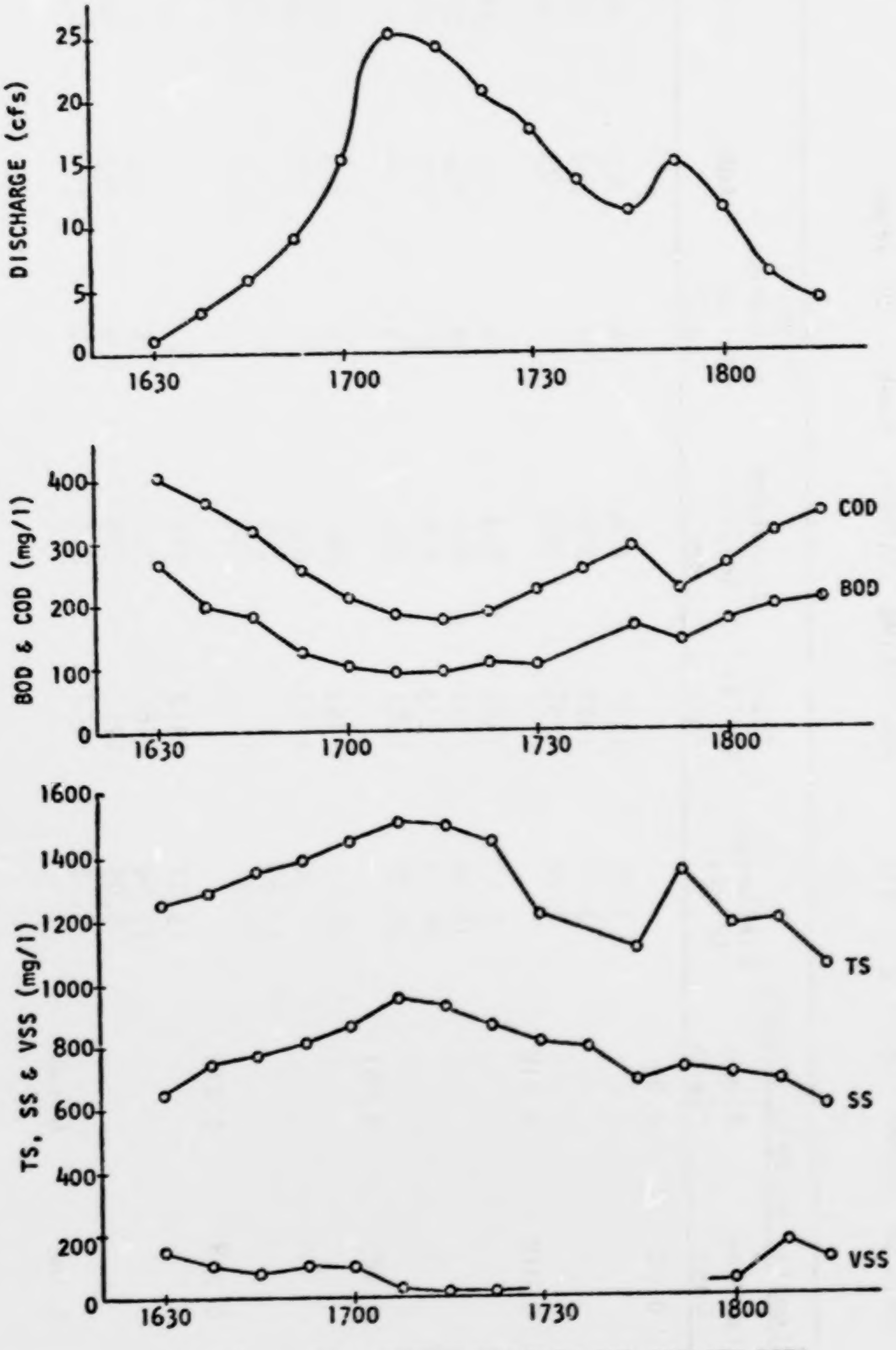


Figure 70. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: APRIL 27, 1970
MONITORING STATION: BANK - 78" SEWER

Table 18 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: April 27, 1970 MONITORING STATION: Outlet - 10' x 15' Sewer

Time	Intensity of Rain (in/hr)		Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	Volatile Suspended Solids mg/l	BOD	COD
	Laidlaw Gage	Ridge Gage						
1620	0.00	0.00						
1650	0.018	0.018						
1657:30			95.0					
1705			108.3	610	480	90	284	658
1712:30			132.3	920	670	200		
1720	0.081	0.081	143.2	1240	920	260	124	184
1727:30			150.6	1410	1060	240		
1735			154.1	1630	1250	340	75	80
1742:30			164.6	1850	1340	360	95	100
1750	0.218	0.218	155.9	1340	1280	320		
1757:30			148.3	1680	1160	270	128	296
1805			139.5	1420	1050	300		
1812:30			136.1	1280	980	340	214	552
1820	0.148	0.148	126.3	1060	920	260		
1827:30			121.5	940	840	330	251	930
1835			120.1	880	820	330		
1842:30			136.5	1350	1070	390	226	756
1850	0.058	0.058	126.1	930	860	290		
1857:30			117.2	720	670	190	292	923
1905			104.3	580	510	100	412	1081

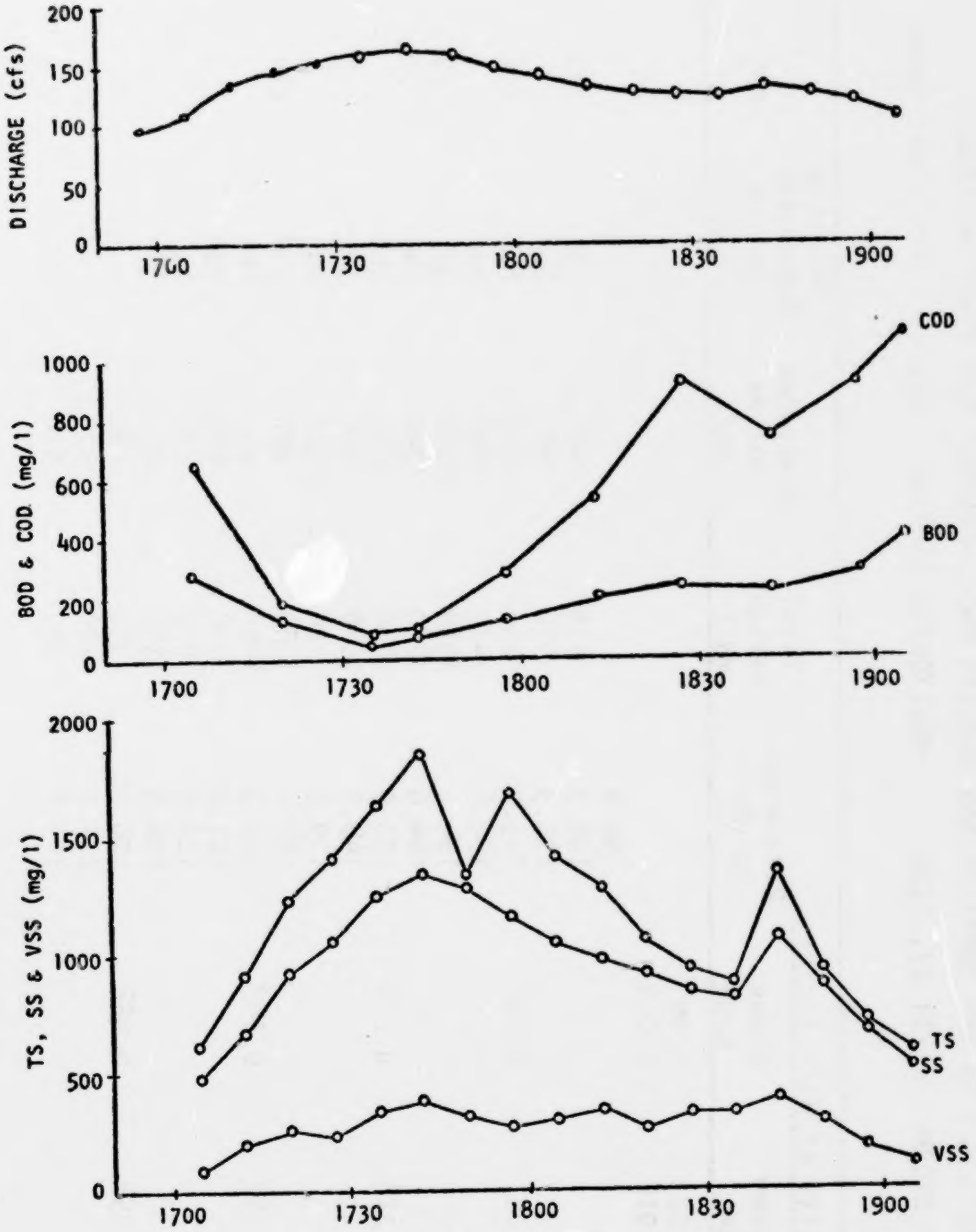


Figure 71. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: APRIL 27, 1970
MONITORING STATION: OUTLET - 10' x 15' SEWER

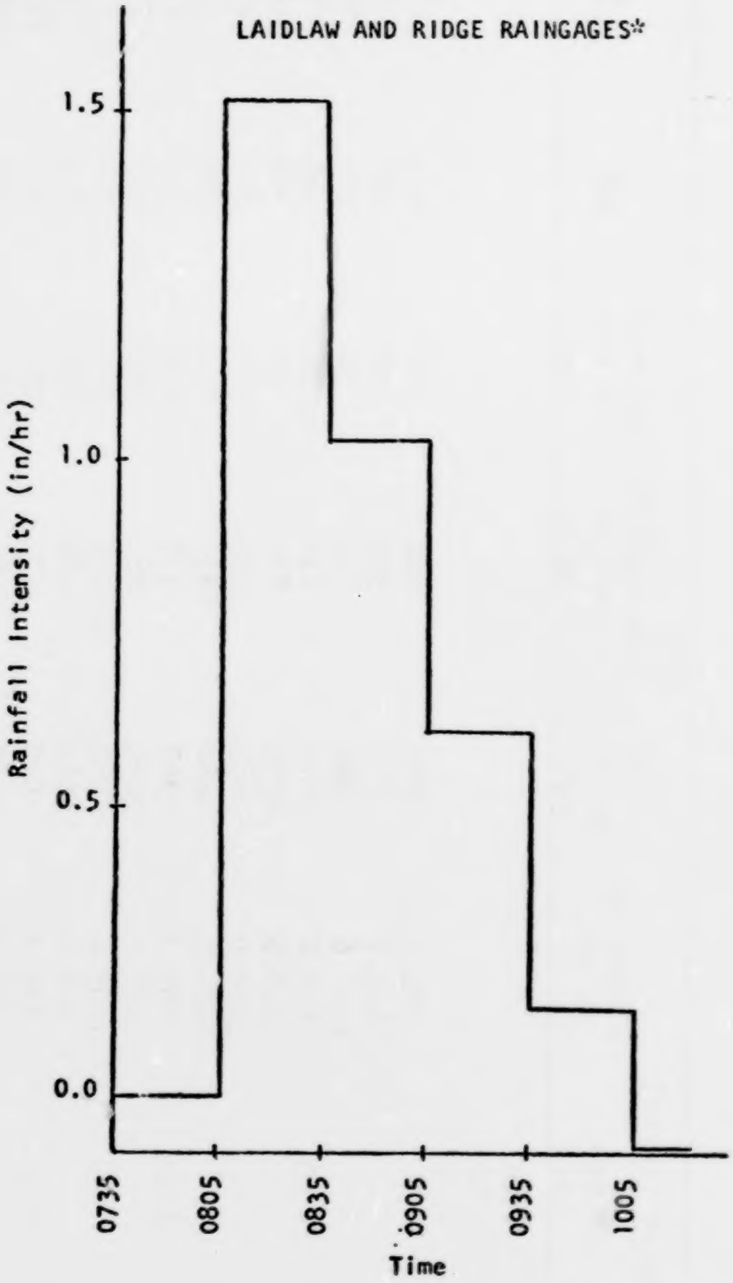


Figure 72. RAIN OF MAY 12, 1970
HYETOGRAPH

*Until August 1971 these two raingages had a 7 day rotation period; therefore, no difference in readings from the charts could be detected.

Table 19 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: May 12, 1970 MONITORING STATION: Outlet - 10' x 15' Sewer

Time	Intensity of Rain (in/hr) Laidlaw Gage	Intensity of Rain (in/hr) Ridge Gage	Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	Volatile Suspended Solids mg/l	BOD	COD
0730	0.00	0.00						
0735	0.08	0.08						
0805	1.52	1.52						
0812:30			106.3	642	480	170	420	104
0820			450.7	715	520	100	230	120
0827:30			1071.6	890	540	90	200	160
0835	1.03	1.03	1874.0	1040	670	50	130	96
0842:30			2828.6	1270	810	30	160	184
0850			2800.0	1680	1150	310	200	456
0857:30			1924.5	1860	1220	270	200	280
0905	0.61	0.61	1762.3	1940	1370	250	150	320
0912:30			1411.7	2010	1460	200	200	320
0920			1109.6	1850	1290	210	205	560
0927:30			928.4	1640	1180	220	320	920
0935	0.21	0.21	625.3	1570	1120	280	350	1544
0942:30			511.3	1350	1060	190	580	1560
0950			403.2	1260	920	270	600	1720
0957:30			291.6	1110	780	270	620	1840
1005	0.005	0.005	106.2	930	510	70	700	1860
1012:30			101.4	780	450	70	700	2000

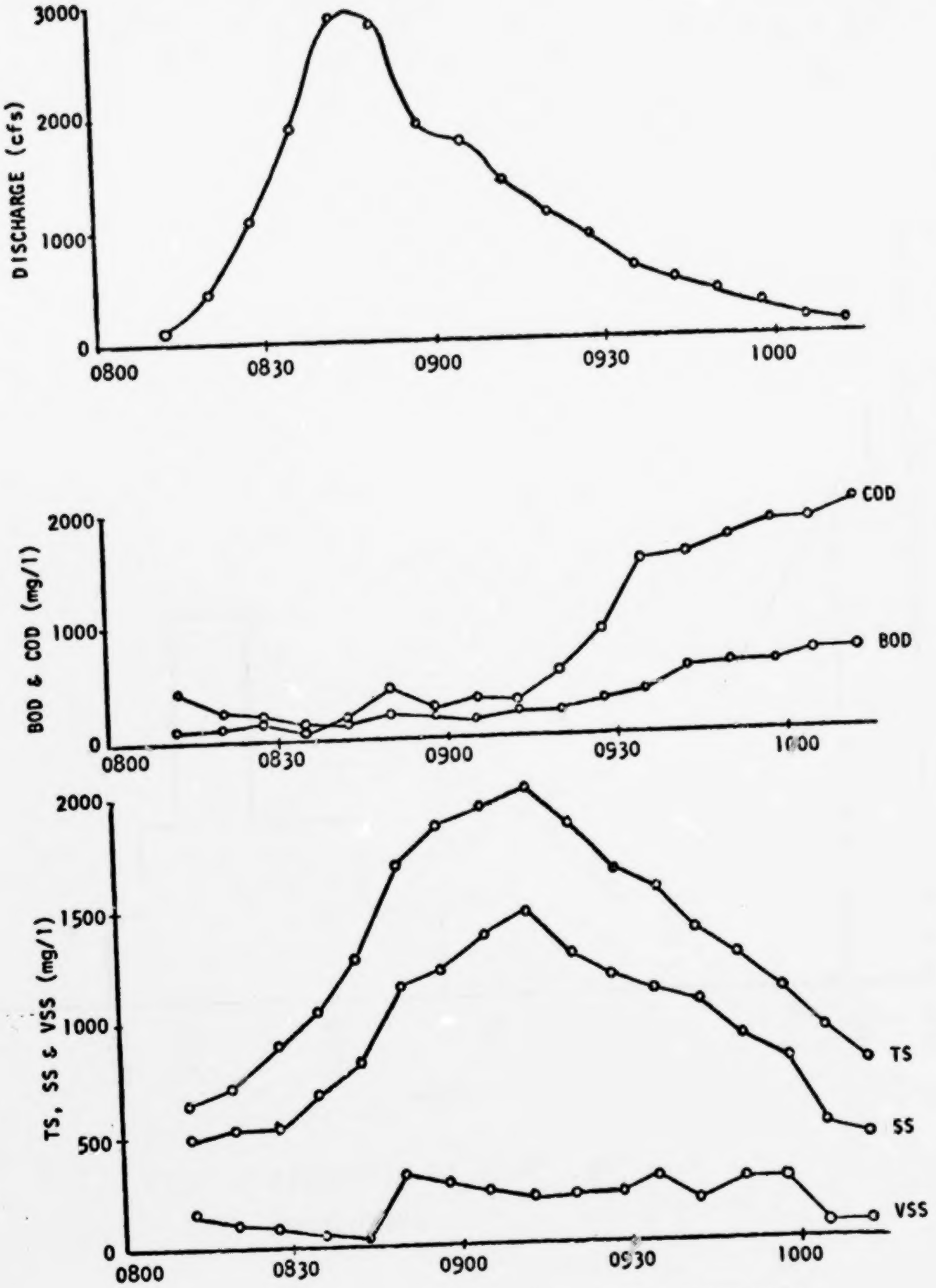


Figure 73. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: May 12, 1970
MONITORING STATION: OUTLET - 10' x 15' SEWER

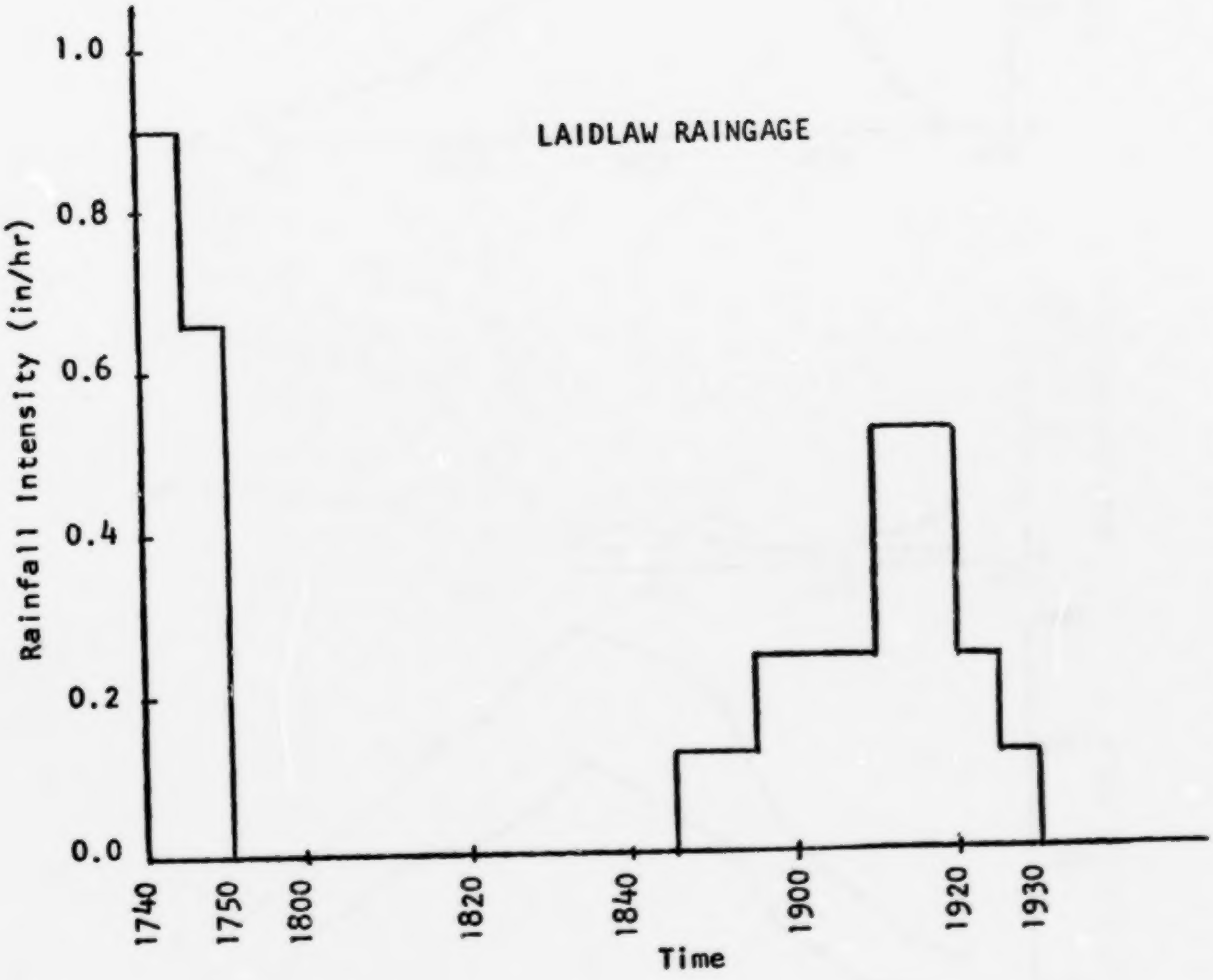


Figure 74. RAIN OF SEPTEMBER 3, 1970
HYETOGRAPH

TABLE 20 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

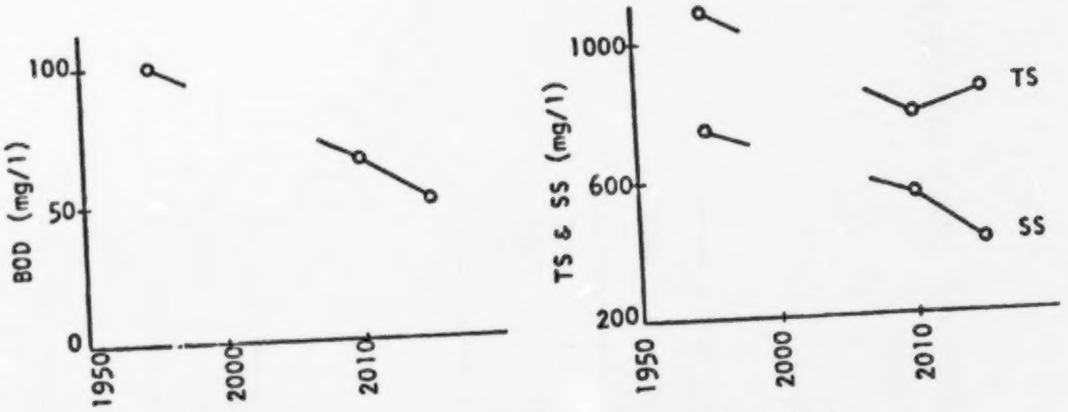
DATE OF STORM: September 3, 1970

(a) MONITORING STATION: Bank - 54" Sewer

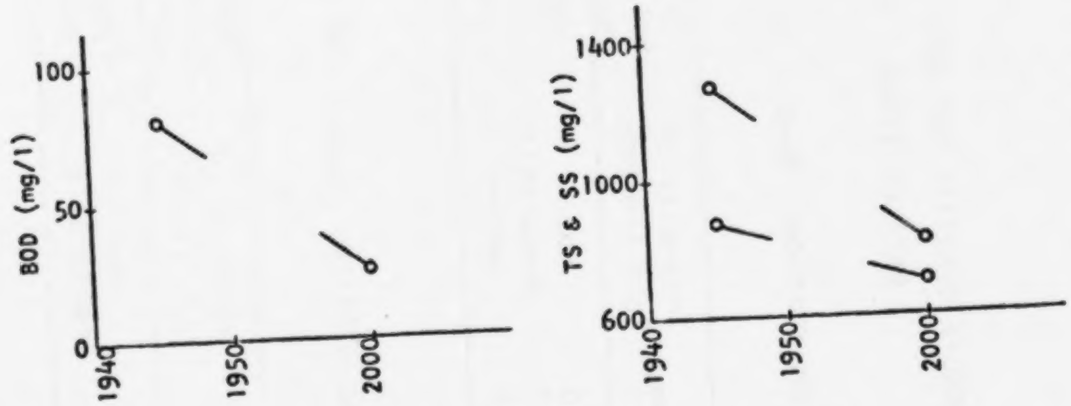
Time	Intensity of Rain (in/hr)	Total Solids mg/l	Suspended Solids mg/l	BOD
1955	not measurable amount	1088	748	100
2010		782	547	65
2015		846	405	50

(b) MONITORING STATION: Bank - 78" Sewer

Time	Intensity of Rain (in/hr)	Total Solids mg/l	Suspended Solids mg/l	BOD
1945	non measurable	1264	869	80
2000		908	696	25



MONITORING STATION: BANK - 54" SEWER



MONITORING STATION: BANK - 78" SEWER

Figure 75. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 3, 1970

Table 21 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: September 3, 1970 MONITORING STATION: Longview - 10' x 15' Sewer

Time	Intensity of Rain (in/hr)	Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	BOD
2000	not measurable amount	63.70	1120	465	115
2005		58.20	876	421	130
2010		48.00	984	348	40
2020		43.28	752	394	40
2025		27.00	688	328	85
2030		23.56	-	286	80

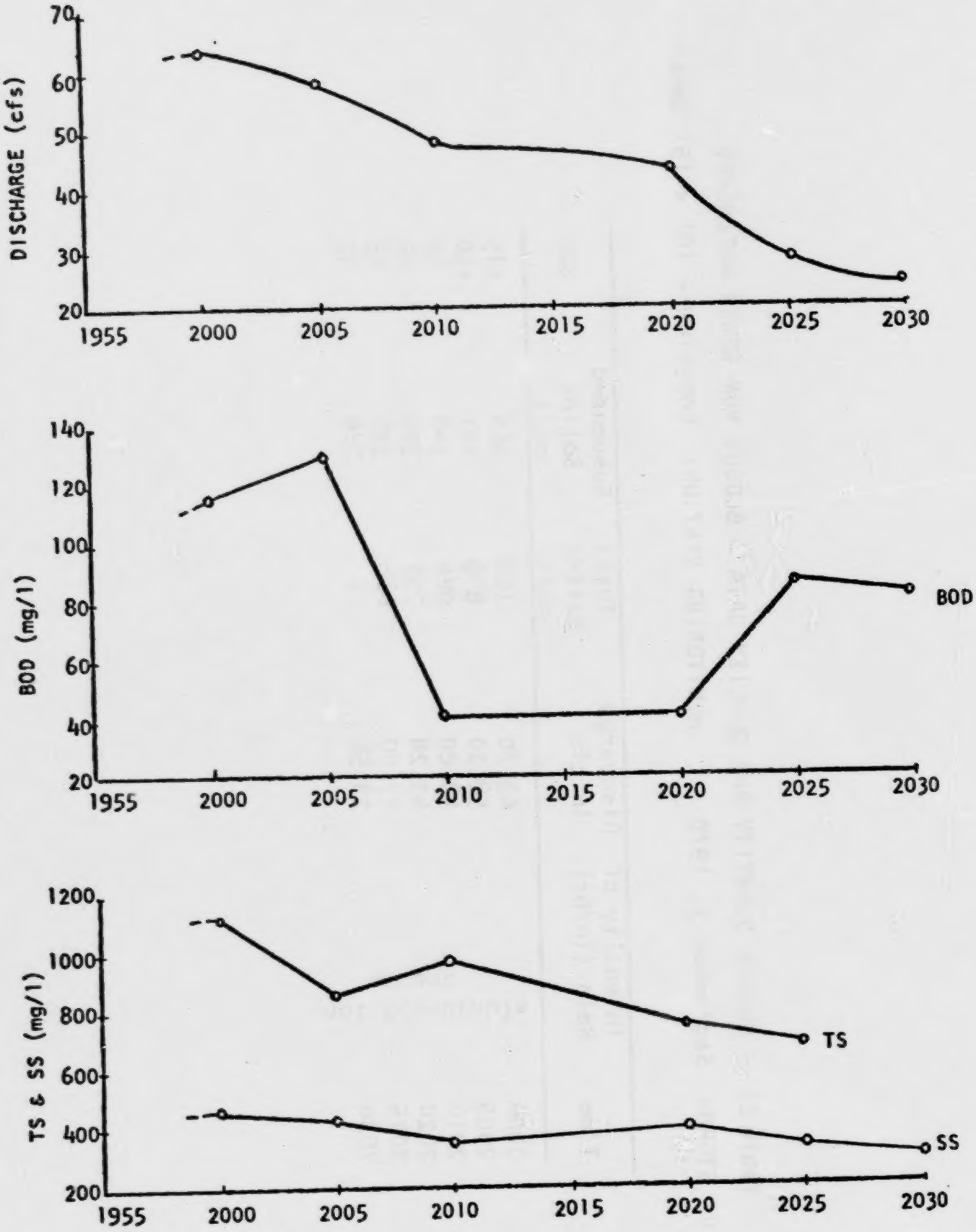


Figure 76. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 3, 1970
MONITORING STATION: LONGVIEW - 10' x 15' SEWER

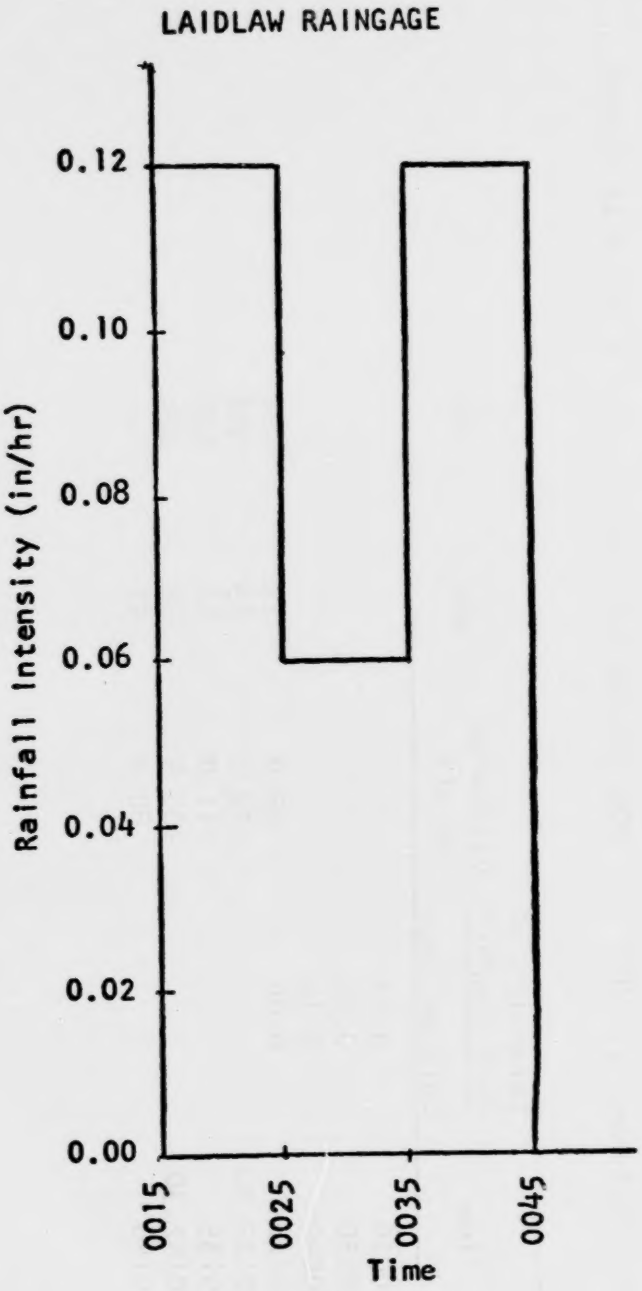


Figure 77 . RAIN OF SEPTEMBER 5, 1970
HYETOGRAPH

Table 22 - RUNOFF QUANTITY AND QUAL!TY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: September 5, 1970 MONITORING STATION: Outlet - 10' x 15' Sewer

Time	Intensity of Rain (in/hr) Laidlaw Gage	Discharge in cfs	BOD	COD
0020	0.12			
0030	0.06			
0040	0.12			
0120	0.00	93.0	77	160
0127:30		122.0	69	328
0135		111.0	47	168
0142:30		108.5	60	204
0150		80.0	57	-

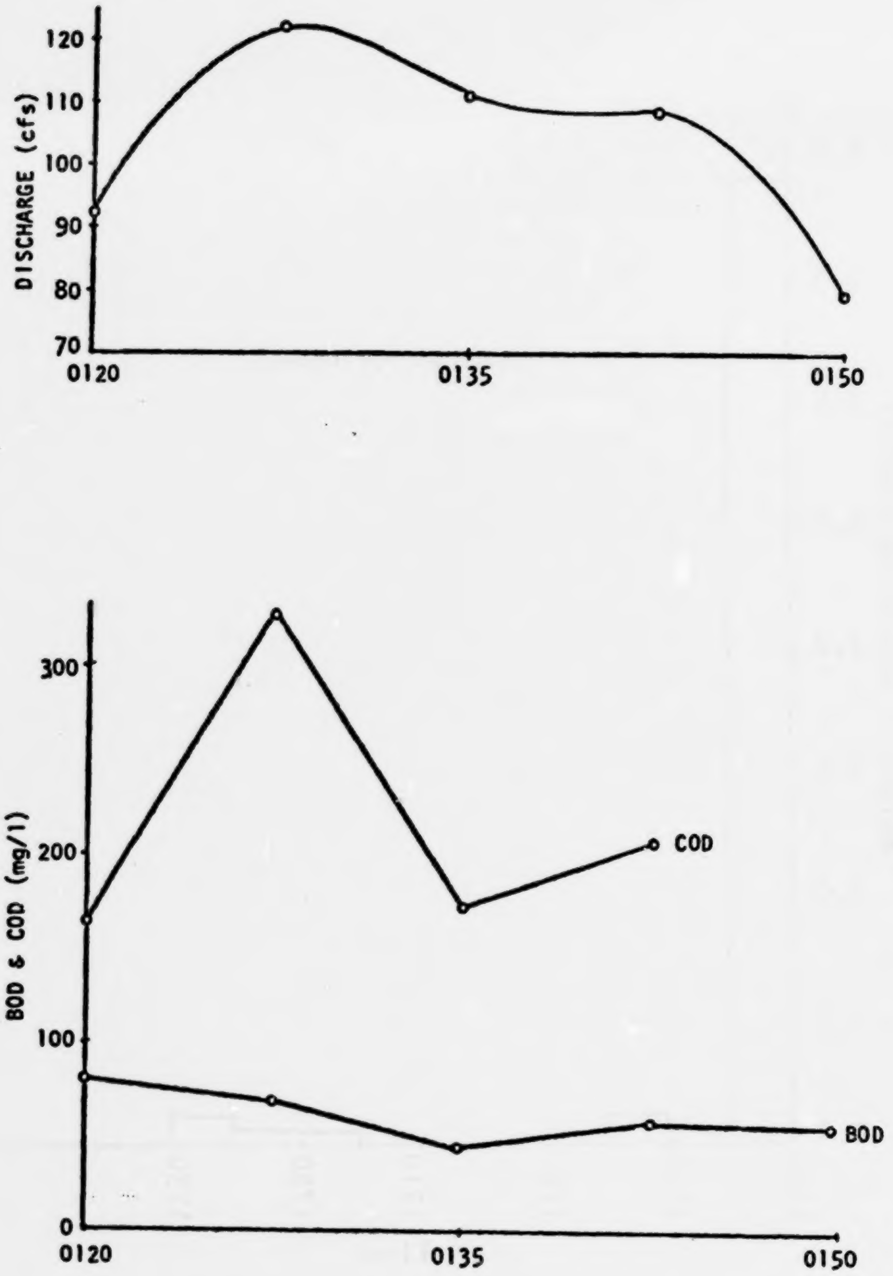


Figure 78. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 5, 1970
MONITORING STATION: OUTLET - 10' x 15' SEWER

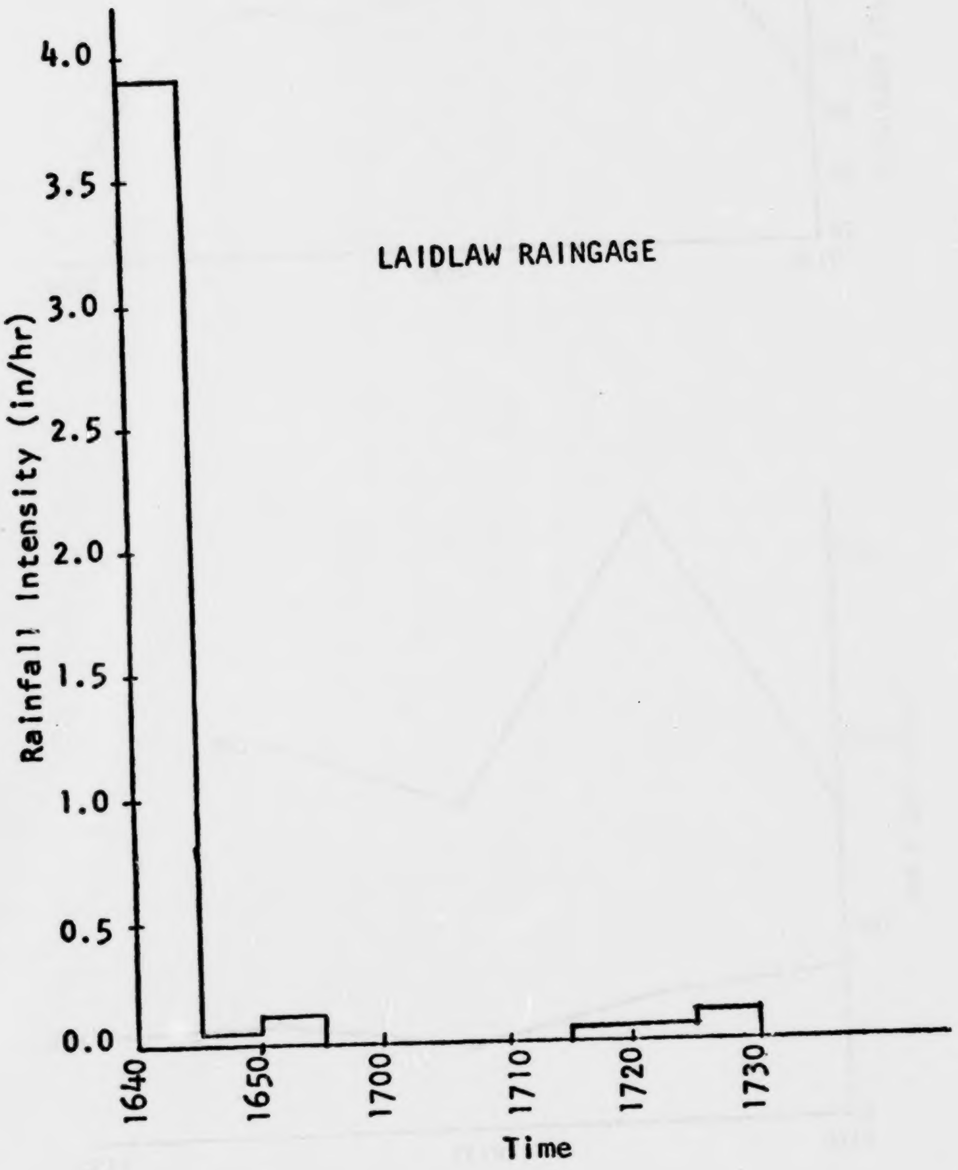


Figure 79 . RAIN OF SEPTEMBER 8, 1970
HYETOGRAPH

Table 23 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: September 8, 1970 MONITORING STATION: Longview - 10' x 15' Sewer

Time	Intensity of Rain (in/hr)	Discharge in cfs	BOD
	Laidlaw Gage		
1640	3.9		
1650	0.12		
1720	0.06		
1730	0.12		
1740	0.00	120	58
1745		104	53
1750		53	54
1755		35	58
1800		21	49
1805		20	60

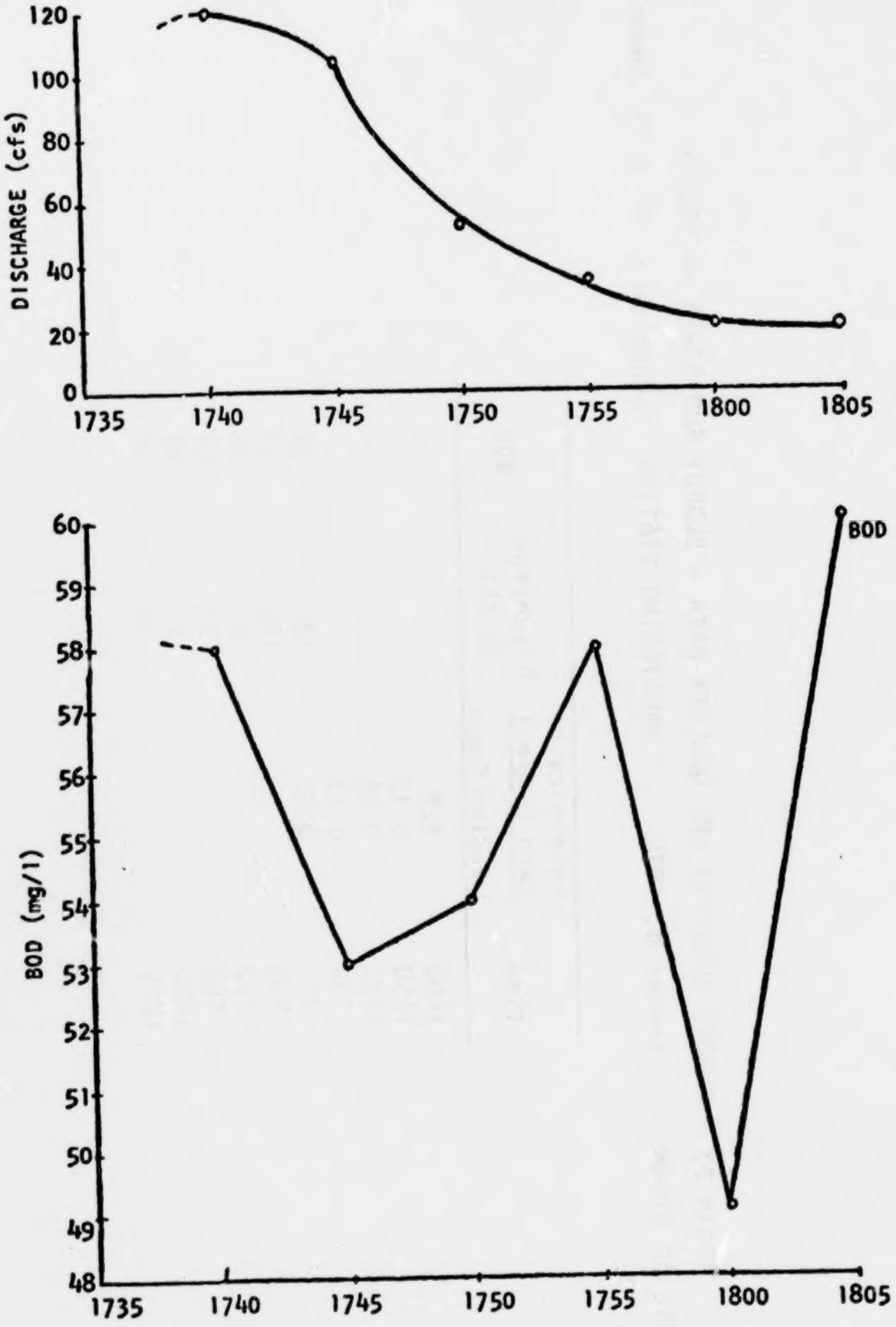


Figure 80. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 8, 1970
MONITORING STATION: LONGVIEW - 10' x 15' SEWER

Table 24 _ RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: September 8, 1970 MONITORING STATION: Outlet - 10' x 15' Sewer

Time	Intensity of Rain (in/hr) Laidlaw Gage	Discharge in cfs	BOD	COD
1650	0.12	85.0	100	276
1657:30	0.00	111.0	40	228
1705	0.00	117.0	54	120
1712:30	0.00	118.0	77	340
1720	0.06	118.0	44	68
1727:30	0.12	119.0	93	-
1735	0.0	119.0	95	348
1742:30		119.0	26	216
1750		119.0	28	320
1757:30		118.0	18	128
1805		118.0	18	140
1812:30		118.0	18	120
1820		88.0	20	-

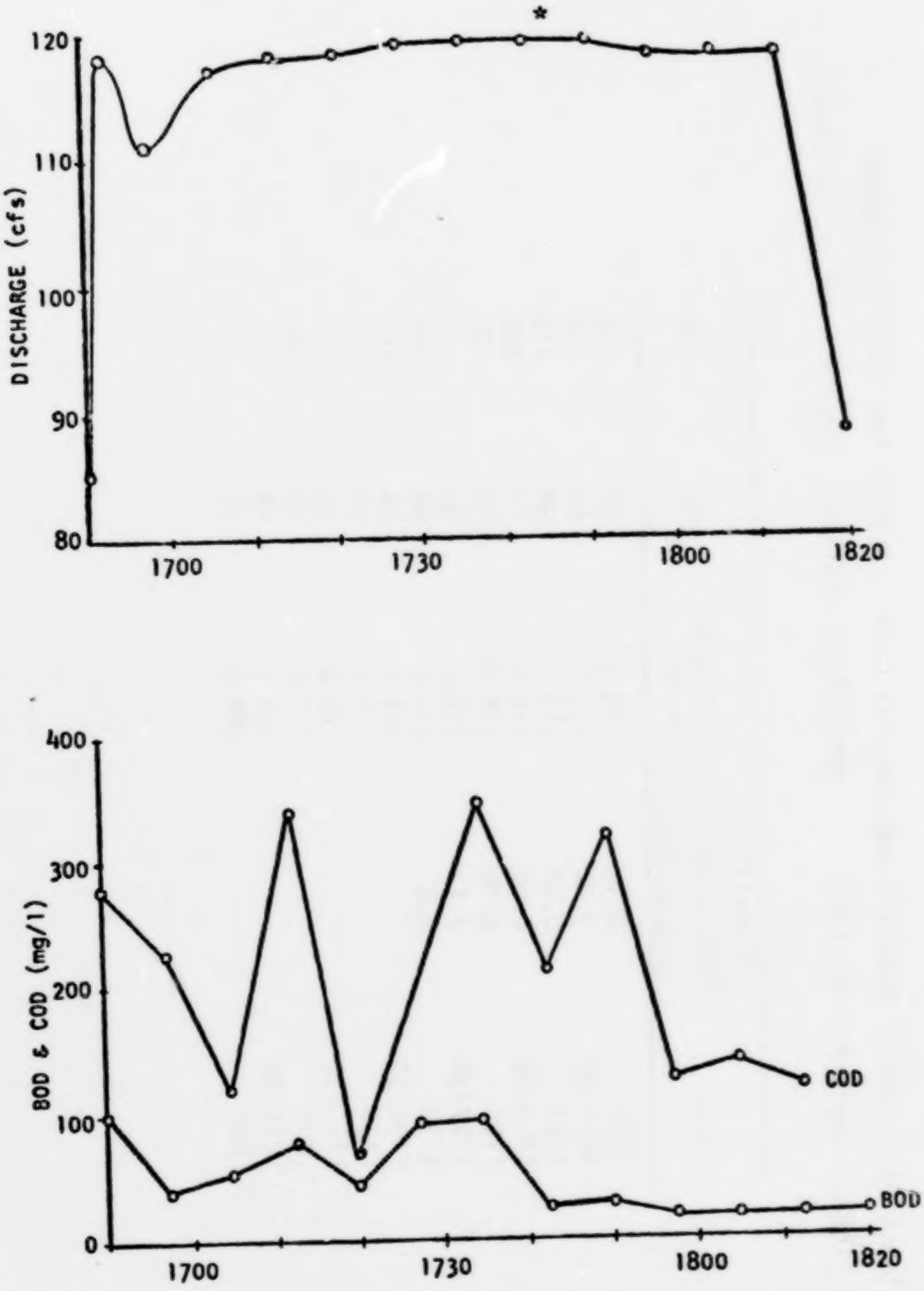
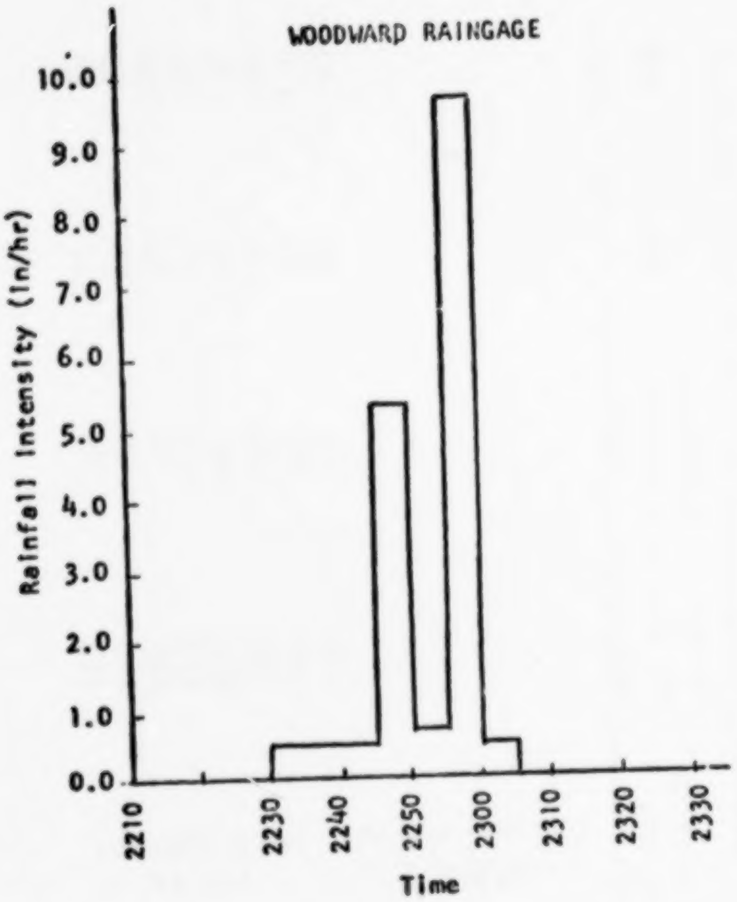


Figure 81. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 8, 1970
MONITORING STATION: OUTLET - 10' x 15' SEWER

*The flat shape of the hydrograph occurred because of a damaged manometer.



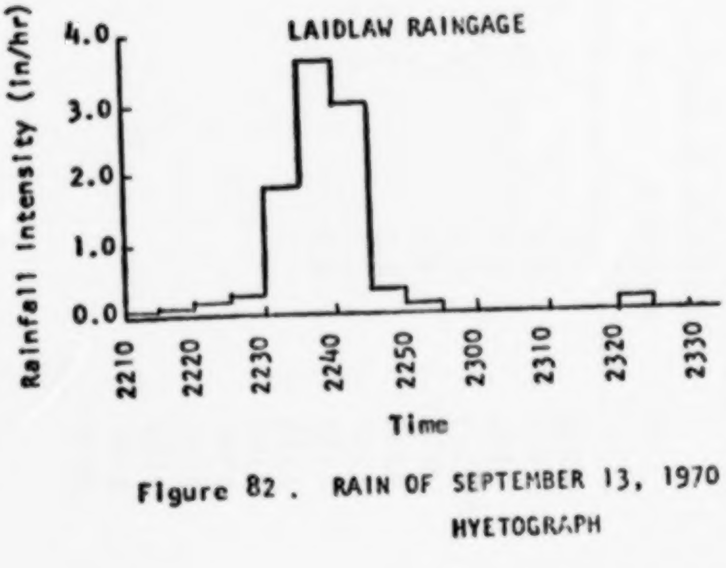


Figure 82 . RAIN OF SEPTEMBER 13, 1970
HYETOGRAPH

Table 25 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: September 13, 1970 MONITORING STATION: Bank - 78'' Sewer

Time	Intensity of Rain (in/hr)		Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	BOD	COD
	Woodward Gage	Laidlaw Gage					
2220		0.18	1.0				
2225	0.0	0.30	7.5				
2230	0.5	1.80	38.0				
2235	0.5	3.6	38.0				
2240	0.5	3.0	6.0				
2245	5.3	0.36	3.0				
2250	0.7	0.12	1.5				
2330	0.0	0.0		416	186	90	132
2335			as dry weather flow	392	162	80	216
2342				365	156	100	140
2345				248	128	100	136
2350				233	112	90	116
2355				228	76	70	208
2400				210	74	70	80
0012				224	72	60	88

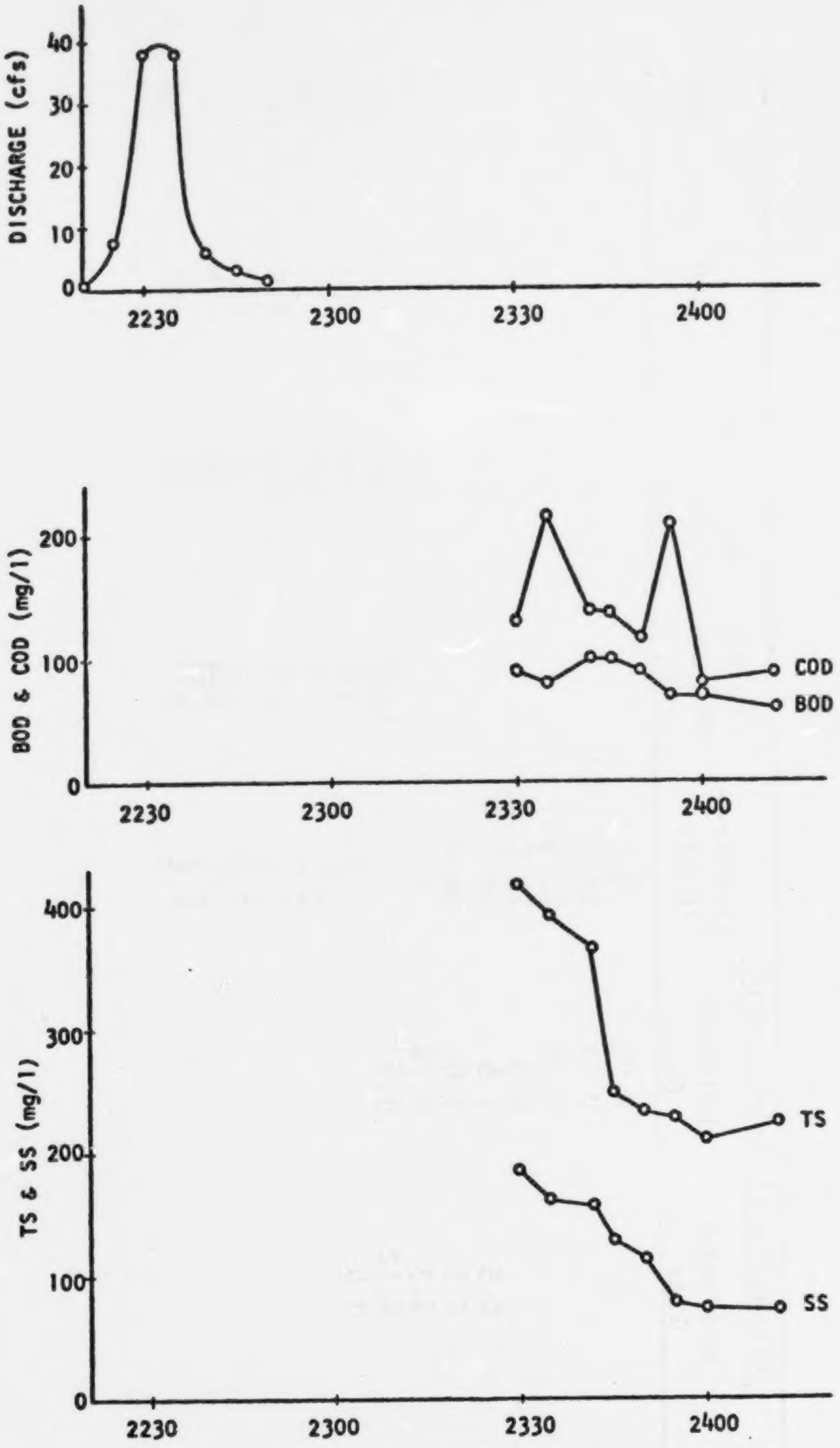


Figure 83. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 13, 1970
MONITORING STATION: BANK - 78" SEWER

Table 26 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: September 13, 1970 MONITORING STATION: Bank - 96" Sewer

Time	Intensity of Rain (in/hr) Woodward Gage	Intensity of Rain (in/hr) Laidlaw Gage	Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	BOD	COD
2210		0.05	3.0				
2215		0.12	22.0				
2220		0.18	52.5				
2225		0.3	21.5				
2230	0.5	1.8	40.0				
2235	0.5	3.6	39.0				
2245	5.3	0.36	4.0				
2325	0.12	0.0		462	224	150	180
2340	0.0		as in dry weather flow	386	148	80	120
2345				299	126	130	140
2350				245	104	640	116
2355				212	98	120	-
2400				206	88	30	-
0010				215	76	110	96

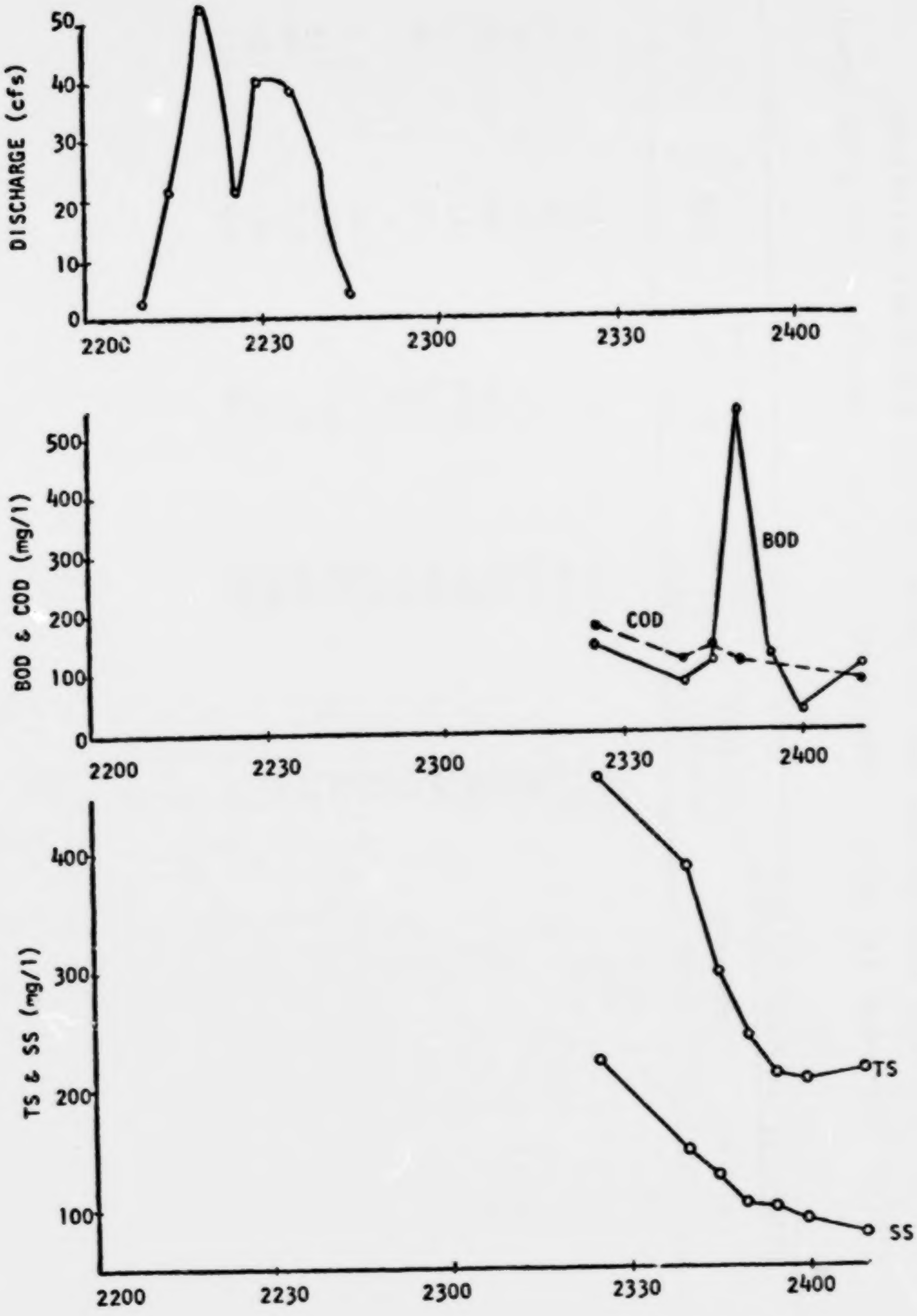


Figure 84. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 13, 1970
MONITORING STATION: BANK - 96" SEWER

Table 27 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: September 13, 1970 MONITORING STATION: Longview - 10' x 15' Sewer

Time	Intensity of Rain (in/hr) Laidlaw Gage	Intensity of Rain (in/hr) Woodward Gage	Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	BOD	COD
2305	0.0	0.5					
2308	0.0	0.0	214	1145	867	141	112
2315	0.0	0.0	112	1066	749	126	112
2320	0.12		90	1042	762	161	136
2325	0.12		58	994	724	140	104
2330	0.0		48	962	682	141	112
2335			46	950	635	95	96
2340			37	874	548	-	-
2345			35	847	512	140	112
2350			32	765	490	175	116
2355			31	698	417	220	144
2400			29	666	424	161	160
0005			27	590	328	166	112

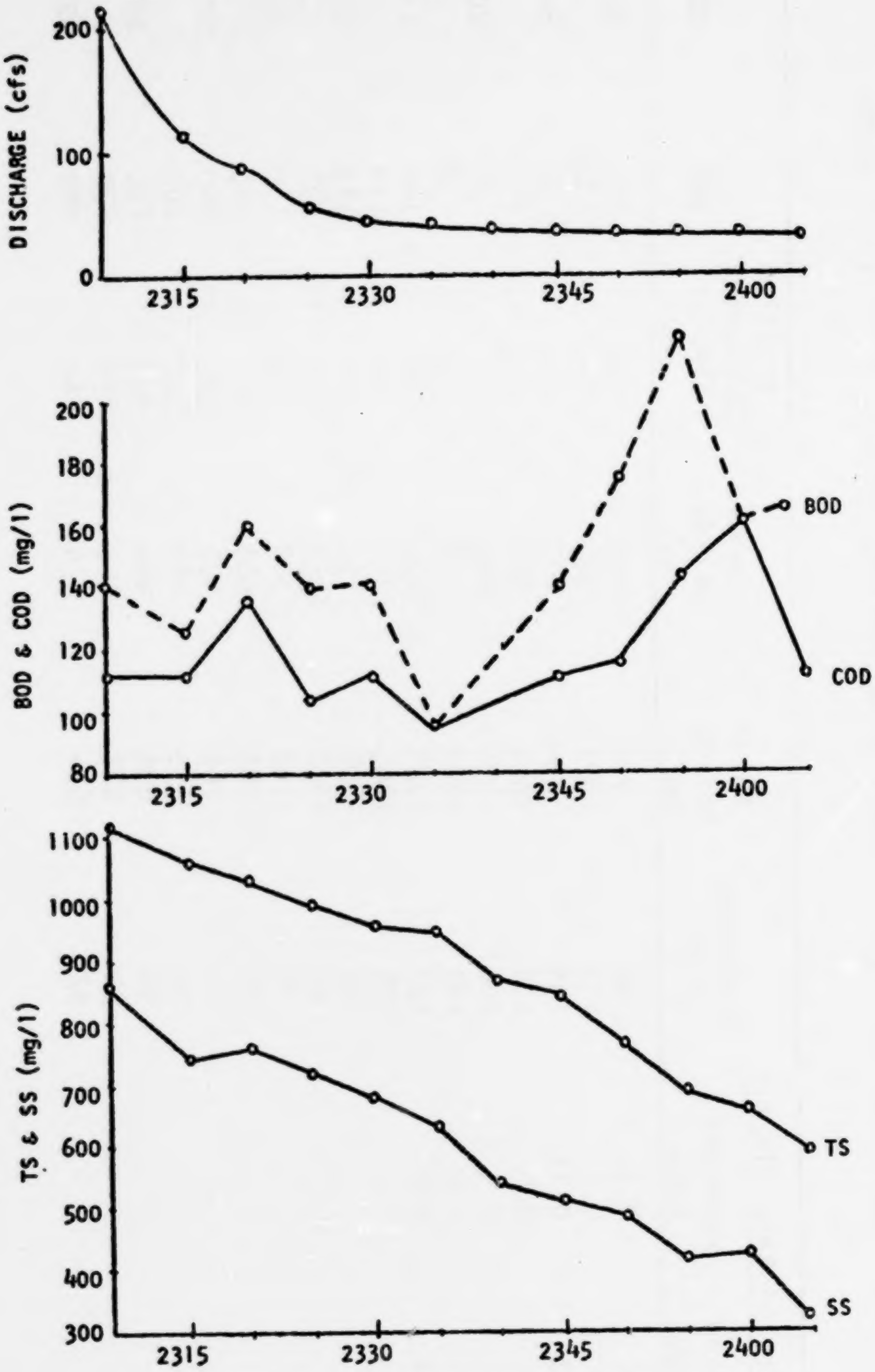


Figure 85. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 13, 1970
MONITORING STATION: LONGVIEW - 10' x 15' SEWER

Table 28 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: September 13, 1970 MONITORING STATION: Outlet - 10' x 15' Sewer

Time	Intensity of Rain (in/hr)		Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	Volatile Suspended Solids mg/l	BOD	COD
	Laidlaw Gage	Woodward Gage						
2235	3.6	0.5	92	717	412	196	270	464
2242:30	3.0	0.5	115	821	564	264	180	
2250	0.12	0.72	135	924	610	302	80	848
2257:30	0.0	9.6	118	1092	840	410	100	
2305	0.0	0.5	106	1436	1232	684	80	208
2312:30	0.0	0.0	107	1705	1408	792	70	
2320	0.12	0.0	107	1785	1426	786	140	240
2327:30	0.0		106	1665	1314	722	110	
2335	0.0		98	1490	1126	623	110	240
2342:30			95	1296	1088	498	130	
2350			96	1140	848	436	110	208
2357:30			96	1002	663	312	140	
0005			95	768	492	273	140	272
0012:30			94	755	480	227	100	
0020			94	696	446	213	120	288
0027:30			94	687	402	195	170	
0035			92	624	374	167	180	328

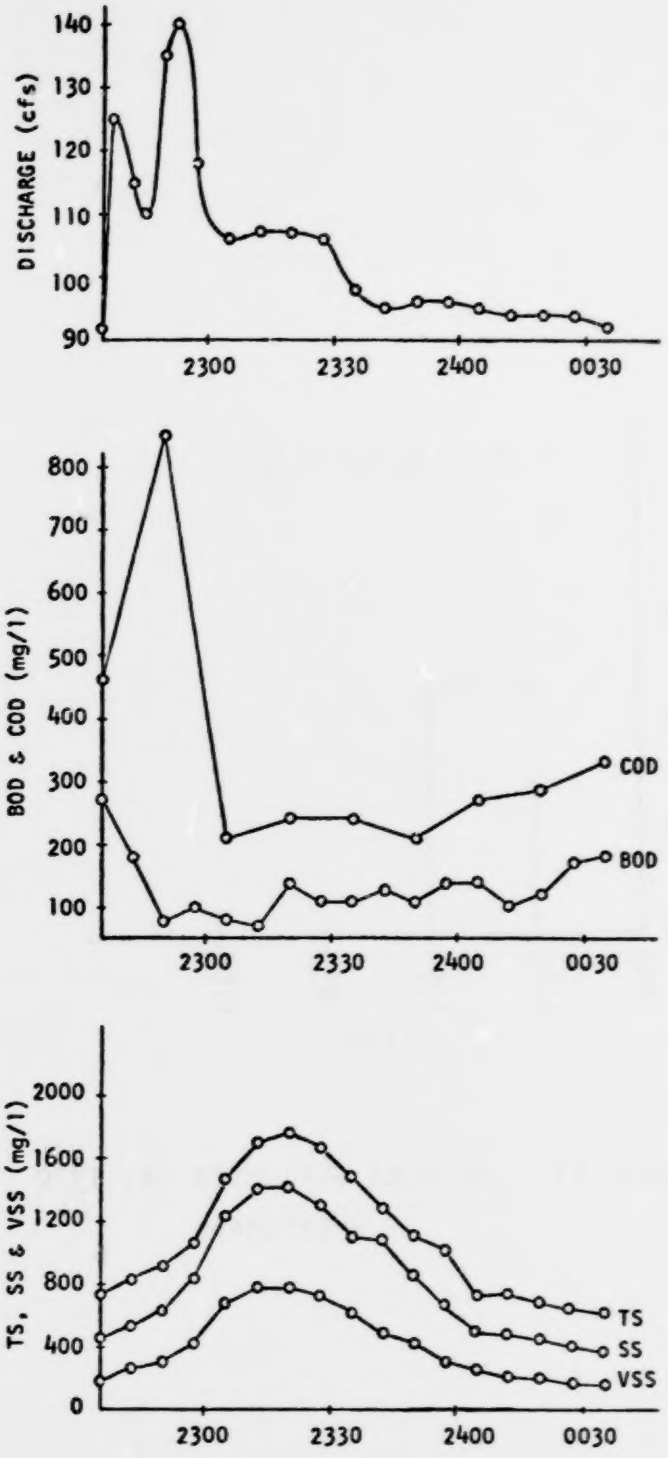


Figure 86. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 13, 1970
MONITORING STATION: OUTLET - 10' x 15' SEWER

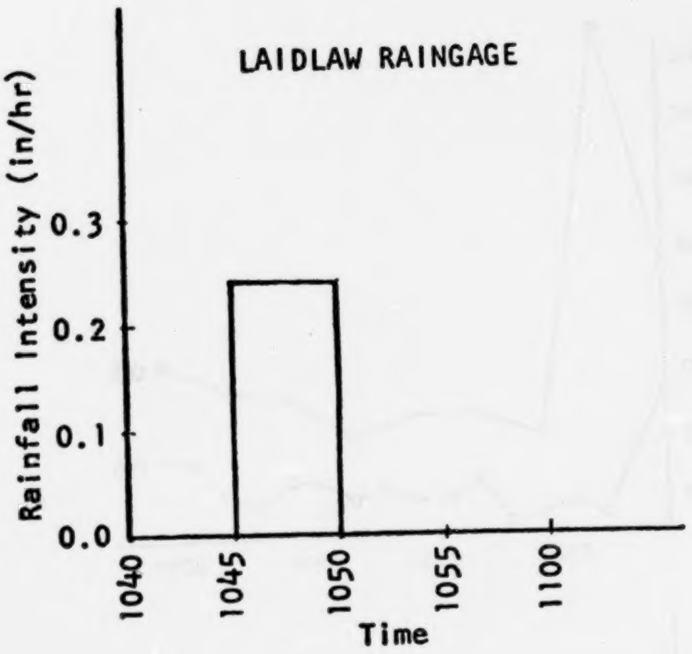


Figure 87 . RAIN OF SEPTEMBER 18, 1970
HYETOGRAPH

Table 29 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: September 18, 1970 MONITORING STATION: Bank - 78" Sewer

Time	Intensity of Rain (in/hr) Laidlaw Gage	Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	BOD	COD
1045	0.24	approximately the dry weather flow				
1050	0.24					
1055	0.00					
1125	0.00		-	-	180	-
1130			-	-	180	728
1135			-	-	190	-
1145			846	580	140	704

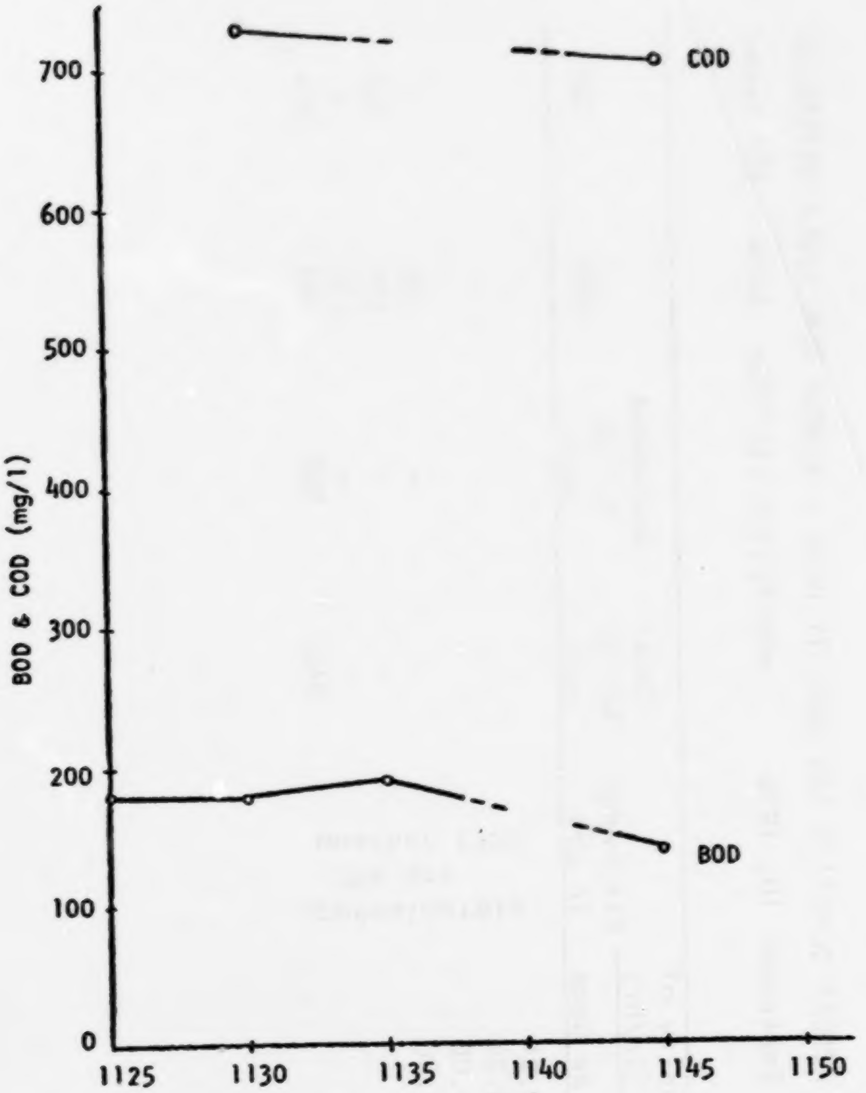


Figure 88. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 18, 1970
MONITORING STATION: BANK - 78" SEWER

Table 30 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: September 18, 1970 MONITORING STATION: Bank - 96" Sewer

Time	Intensity of Rain (in/hr) Laidlaw Gage	Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	BOD	COD
1045	0.24	approximately the dry weather flow				
1050	0.24					
1055	0.0					
1132	0.0		482	392	170	664
1135			688	598	190	780
1145			984	768	100	696
1155			1890	1660	160	372
1205			1160	856	130	332
1215			1064	840	-	272

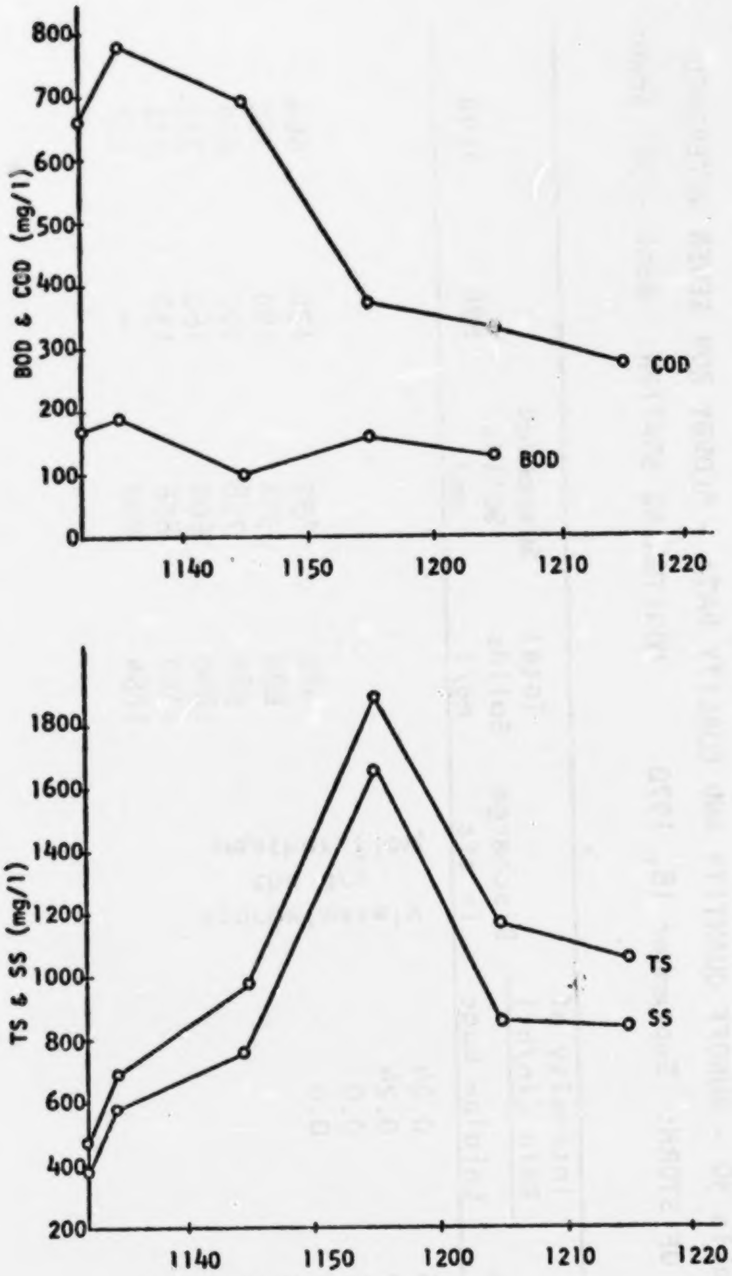


Figure 89. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 18, 1970
MONITORING STATION: BANK - 96" SEWER

Table 31 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: September 18, 1970 MONITORING STATION: Longview - 10' x 15' Sewer

Time	Intensity of Rain (in/hr) Laidlaw Gage	Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	BOD	COD
1045	0.24	approximately the dry weather flow				
1050	0.24					
1055	0.00					
1145			249	178	490	480
1150			228	166	270	600
1200			186	144	190	368
1210			264	180	190	528

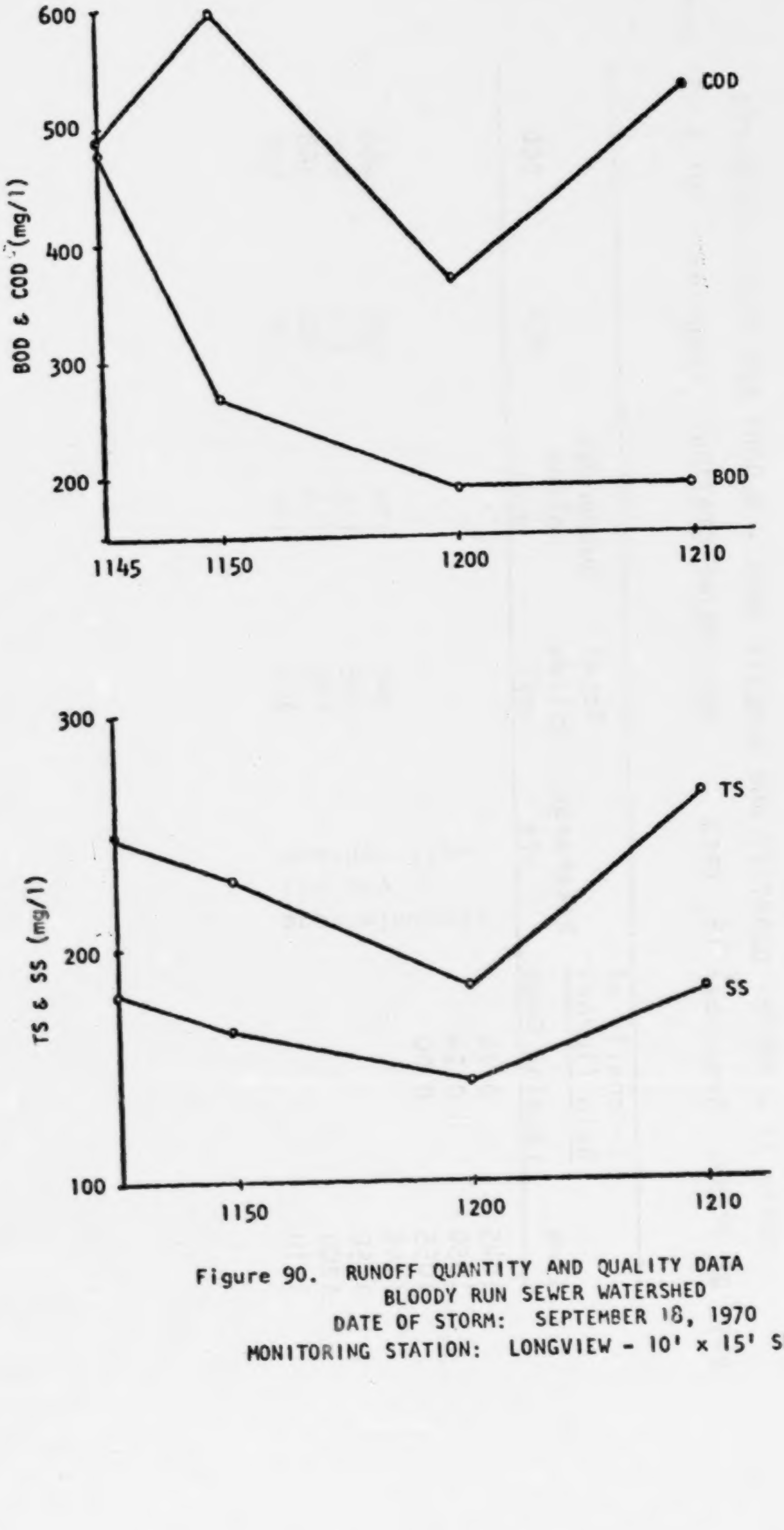


Figure 90. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 18, 1970
MONITORING STATION: LONGVIEW - 10' x 15' SEWER

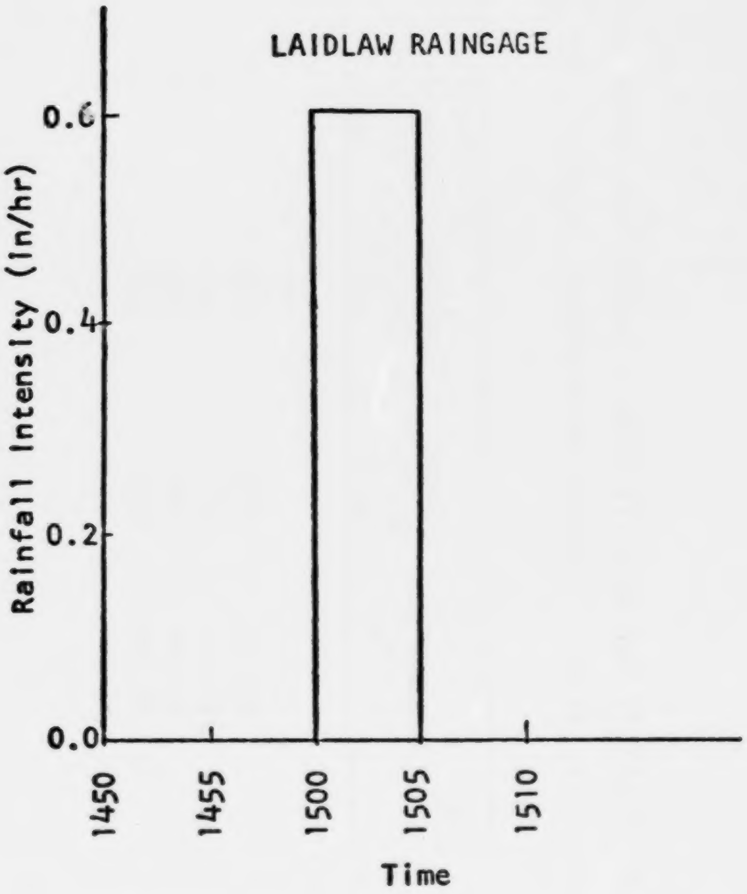


Figure 91. RAIN OF SEPTEMBER 21, 1970
HYETOGRAPH*

*Ridge Raingage indicated a non-measurable amount of rain.

Table 32 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: September 21, 1970 MONITORING STATION: Outlet - 10' x 15' Sewer

Time	Intensity of Rain (in/hr)		Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	Volatile Suspended Solids mg/l	BOD	COD
	Laidlaw Gage	Ridge Gage						
1500	0.60	Non-measurable amount of rain						
1505	0.60							
1548	0.00		85.00	749	468	212	170	468
1555:30			105.00	671	324	186	150	408
1603			97.00	528	286	174	110	428
1610:30			88.00	624	362	170	170	336

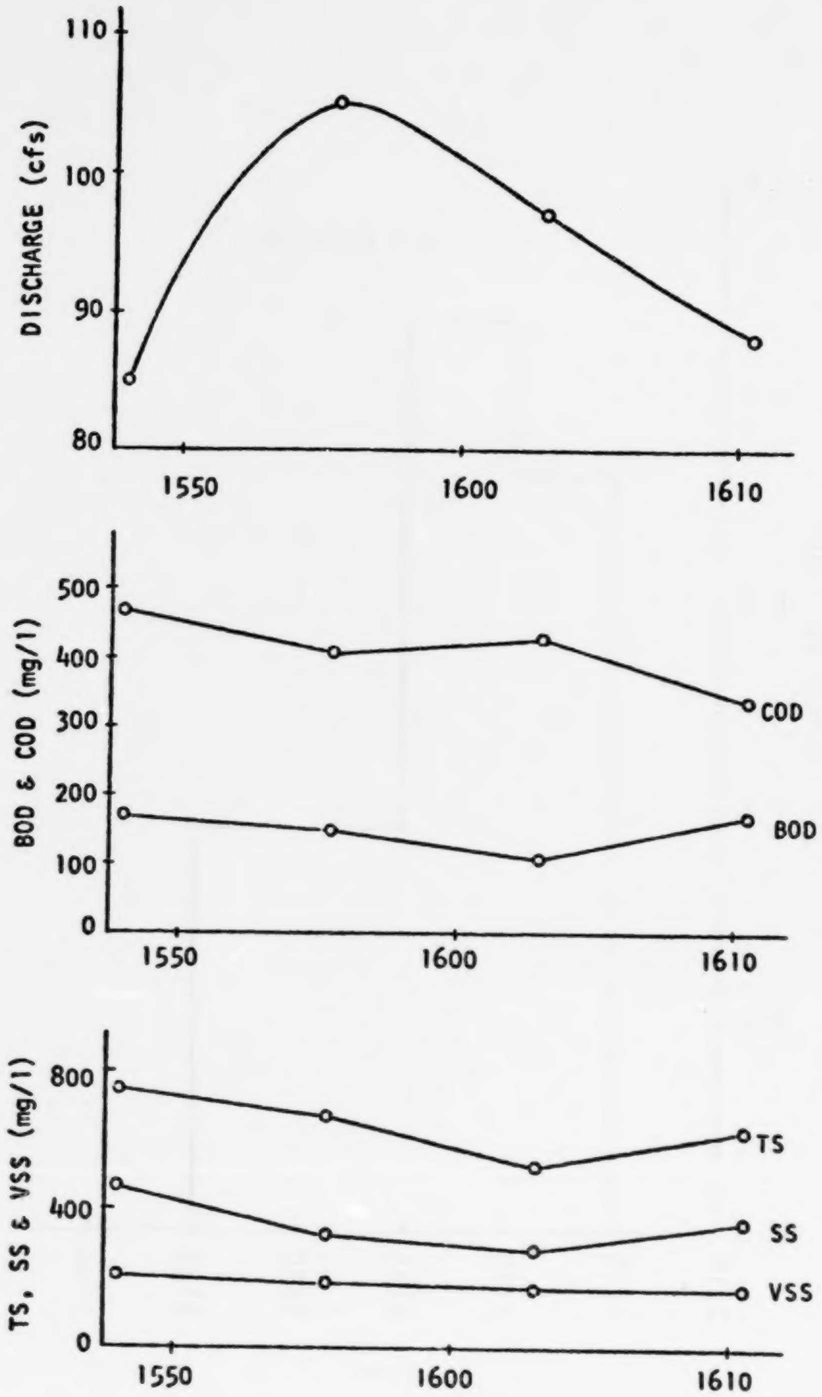


Figure 92. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 21, 1970
MONITORING STATION: OUTLET - 10' x 15' SEWER

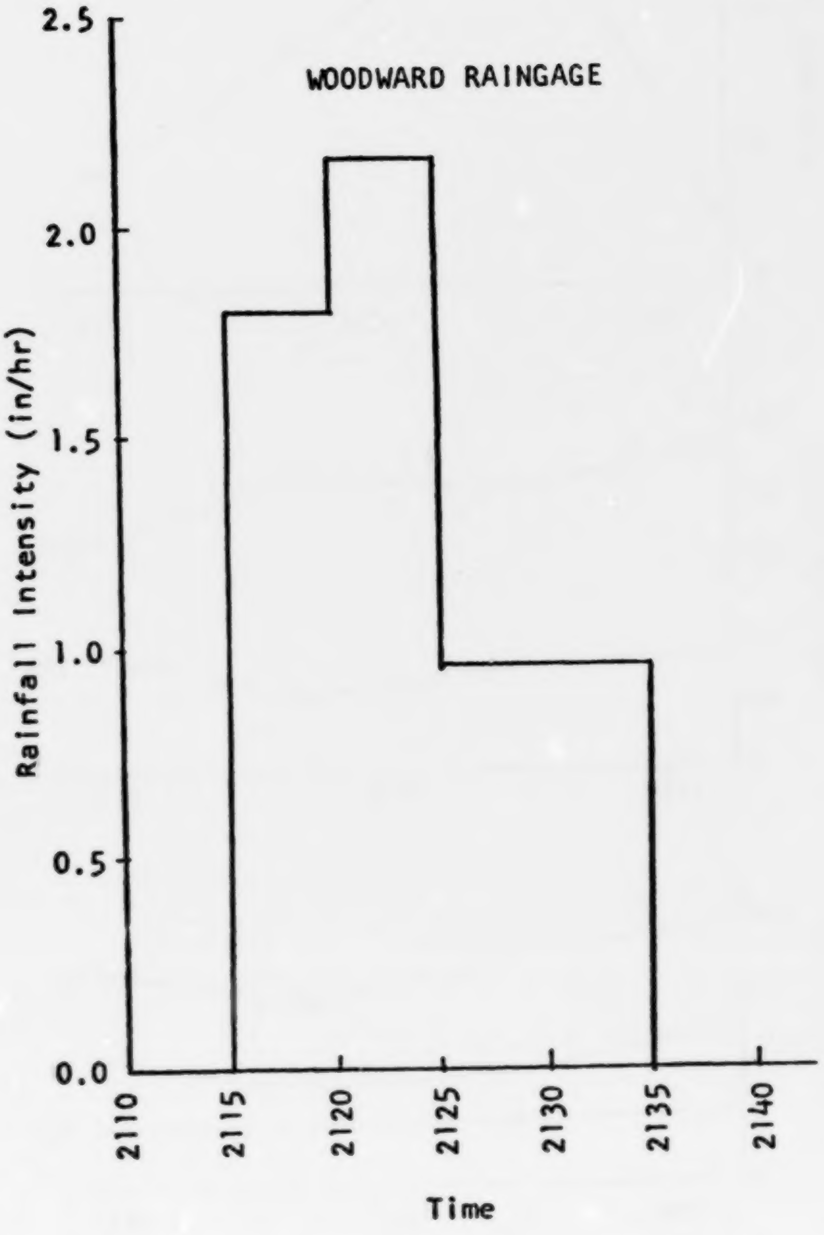


Figure 93 . RAIN OF SEPTEMBER 22, 1970
HYETOGRAPH

Table 33 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: September 22, 1970 MONITORING STATION: Bank - 96" Sewer

Time	Intensity of Rain (in/hr) Woodward Gage	Total Solids mg/l	Suspended Solids mg/l	BOD	COD
2115	1.80				
2120	2.16				
2130	0.96				
2145	0.00	427	110	25	172
2150		480	182	55	156
2155		924	690	75	96
2200		587	208	65	192
2205		-	-	120	176
2210		527	246	130	180
2215		431	176	70	204
2220		362	124	70	180
2230		316	92	55	168
2240		394	148	170	460
2250		384	76	105	400
2300		561	262	65	288

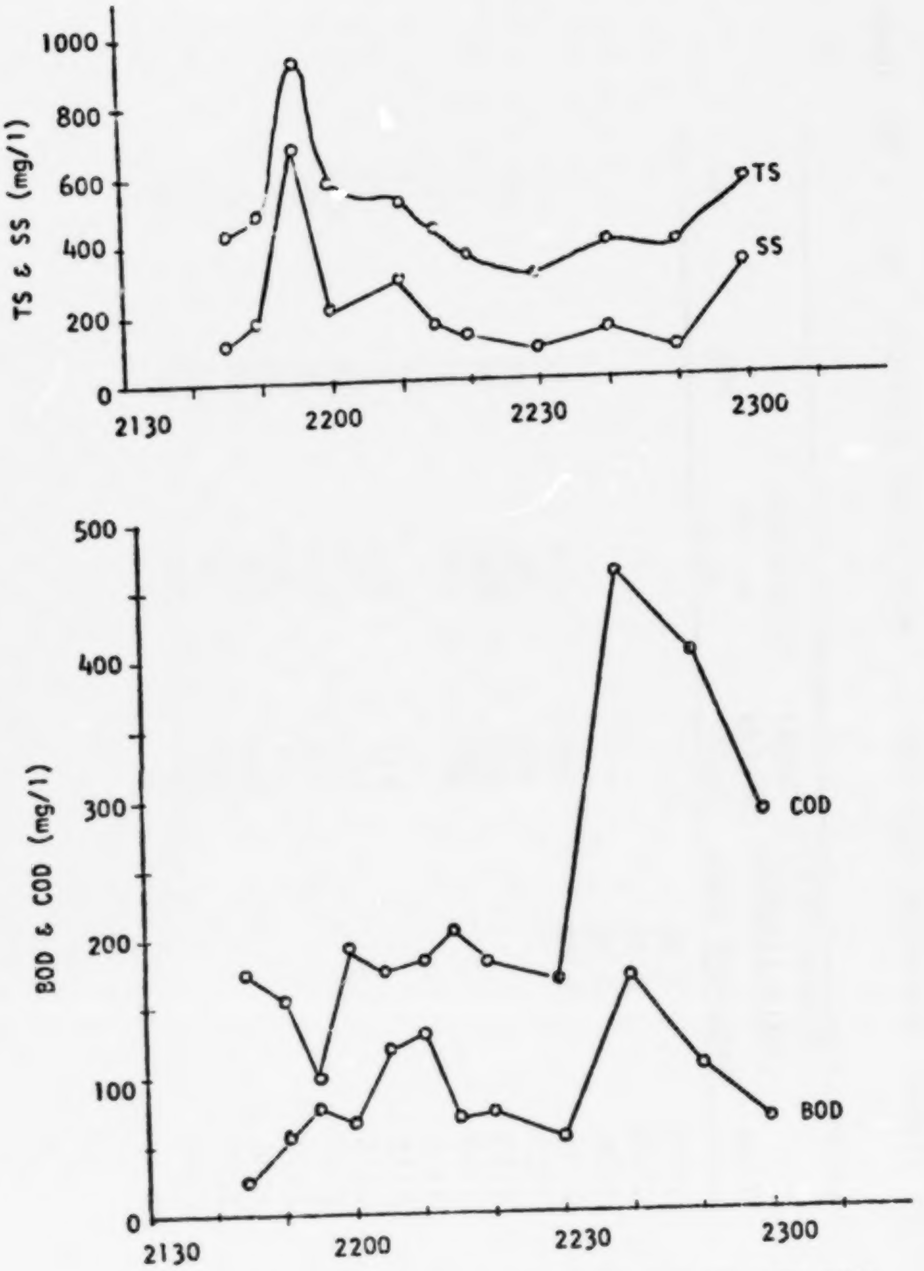


Figure 94. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 22, 1970
MONITORING STATION: BANK - 96" SEWER

Table 34 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: September 22, 1970 MONITORING STATION: Longview-10' x 15' Sewer

Time	Intensity of Rain (in/hr) Woodward Gage	Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	BOD	COD
2115	1.80					
2120	2.16					
2130	0.96					
2155	0.00	58.0	413	166	73	90
2200		30.0	427	118	65	112
2205		29.0	407	128	78	224
2210		30.0	384	102	40	138
2215		20.0	392	116	58	156
2220		15.0	366	108	50	160
2230		12.0	398	160	128	616
2255		10.0	216	132	115	393
2300		10.0	413	182	222	676

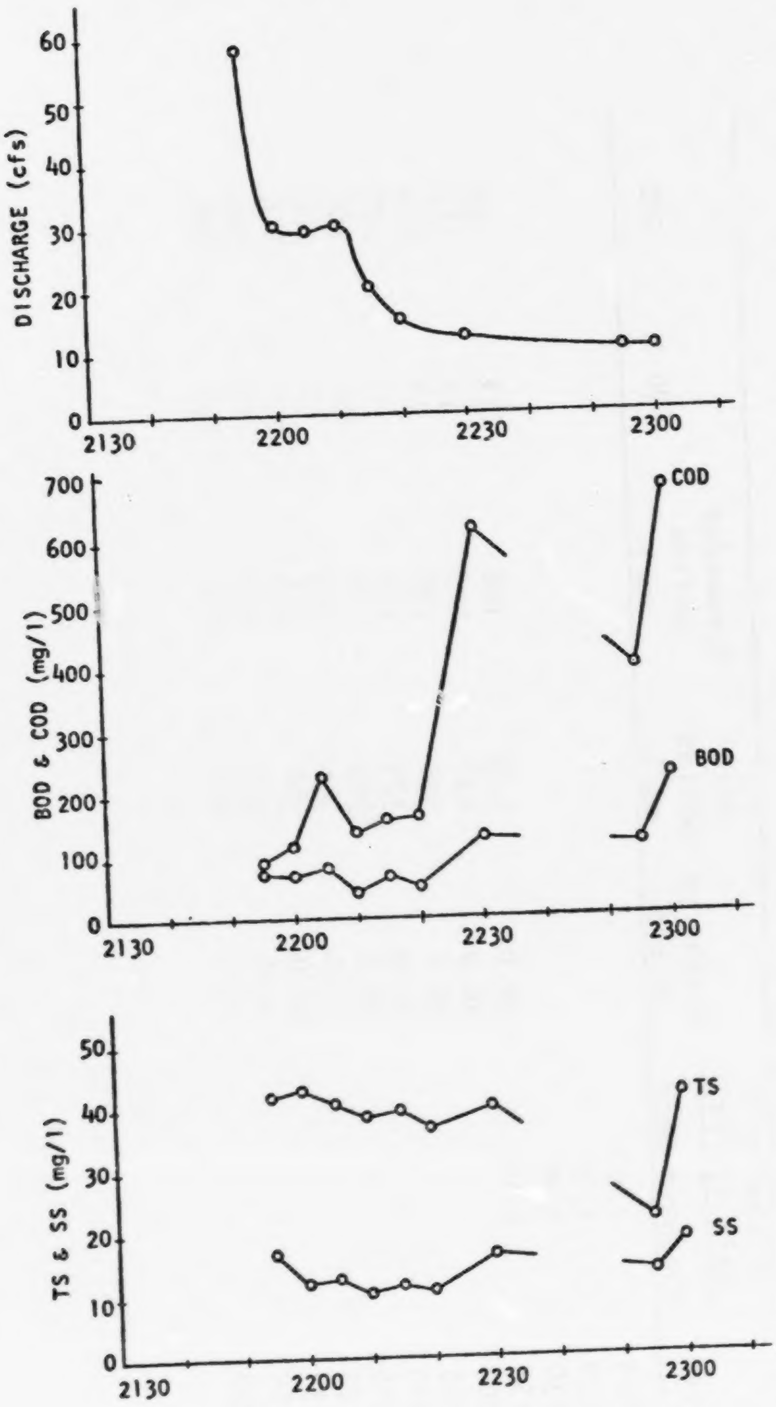


Figure 95. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 22, 1970
MONITORING STATION: LONGVIEW - 10' x 15' SEWER

Table 35 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: September 22, 1970 MONITORING STATION: Outlet - 10' x 15' Sewer

Time	Intensity of Rain (in/hr) Woodward Gage	Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	BOD	COD
2115	1.80					
2120	2.16					
2130	0.96					
2140	0.00	90.0	986	632	160	208
2147:30		120.0	754	484	130	116
2155		107.5	637	441	100	357
2202:30		106.0	1045	712	120	144
2210		105.0	-	-	-	-
2217:30		104.0	821	564	160	693
2225		97.0	878	596	180	442
2232:30		93.0	804	468	150	352
2240		97.5	633	296	170	352
2247:30		77.0	982	608	200	250

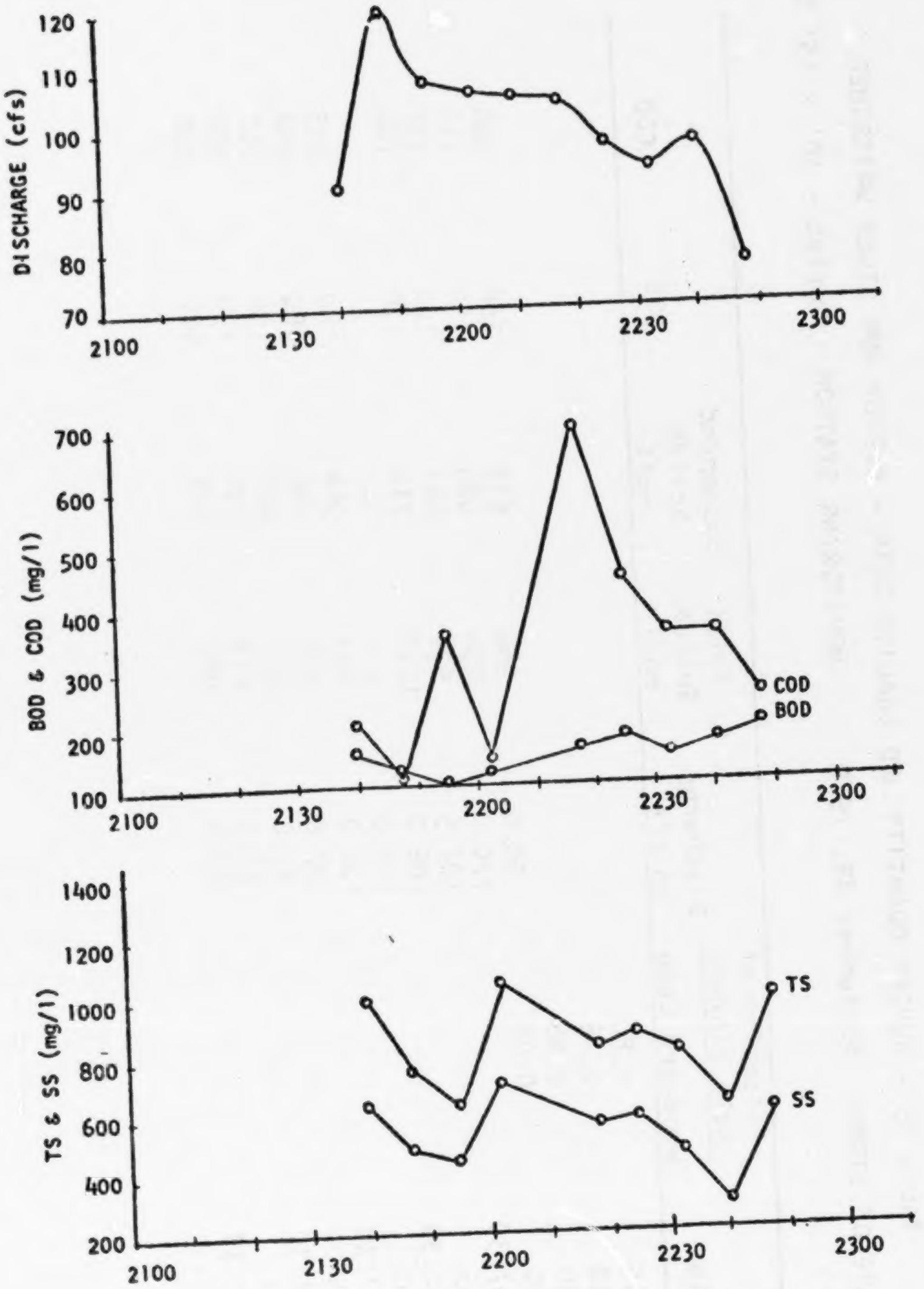


Figure 96. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 22, 1970
MONITORING STATION: OUTLET - 10' x 15' SEWER

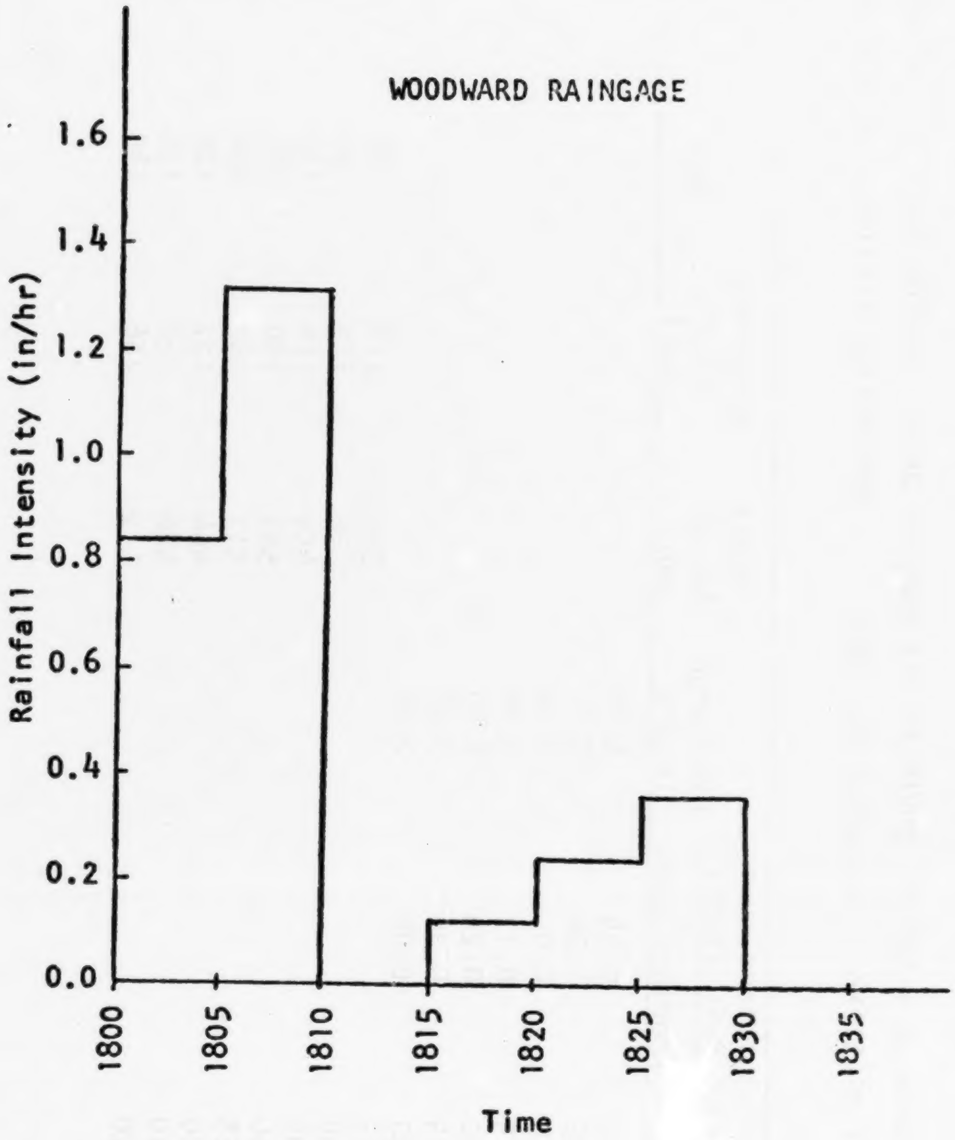


Figure 97 . RAIN OF SEPTEMBER 24, 1970
HYETOGRAPH

Table 36 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: September 24, 1970 MONITORING STATION: Bank - 96'' Sewer

Time	Intensity of Rain (in/hr) Woodward Gage	Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	BOD	COD
1800	0.84	3.00				
1805	1.32	12.50				
1810	0.0	18.00				
1815	0.12	9.00				
1820	0.24	3.50				
1825	0.36	3.00				
1830	0.00	2.00				
1835			519	218	115	324
1840			476	178	95	316
1845			454	124	110	340
1850			567	290	135	452
1855			512	162	430	292
1900			404	122	100	308
1910			426	168	135	396
1920			412	156	140	460

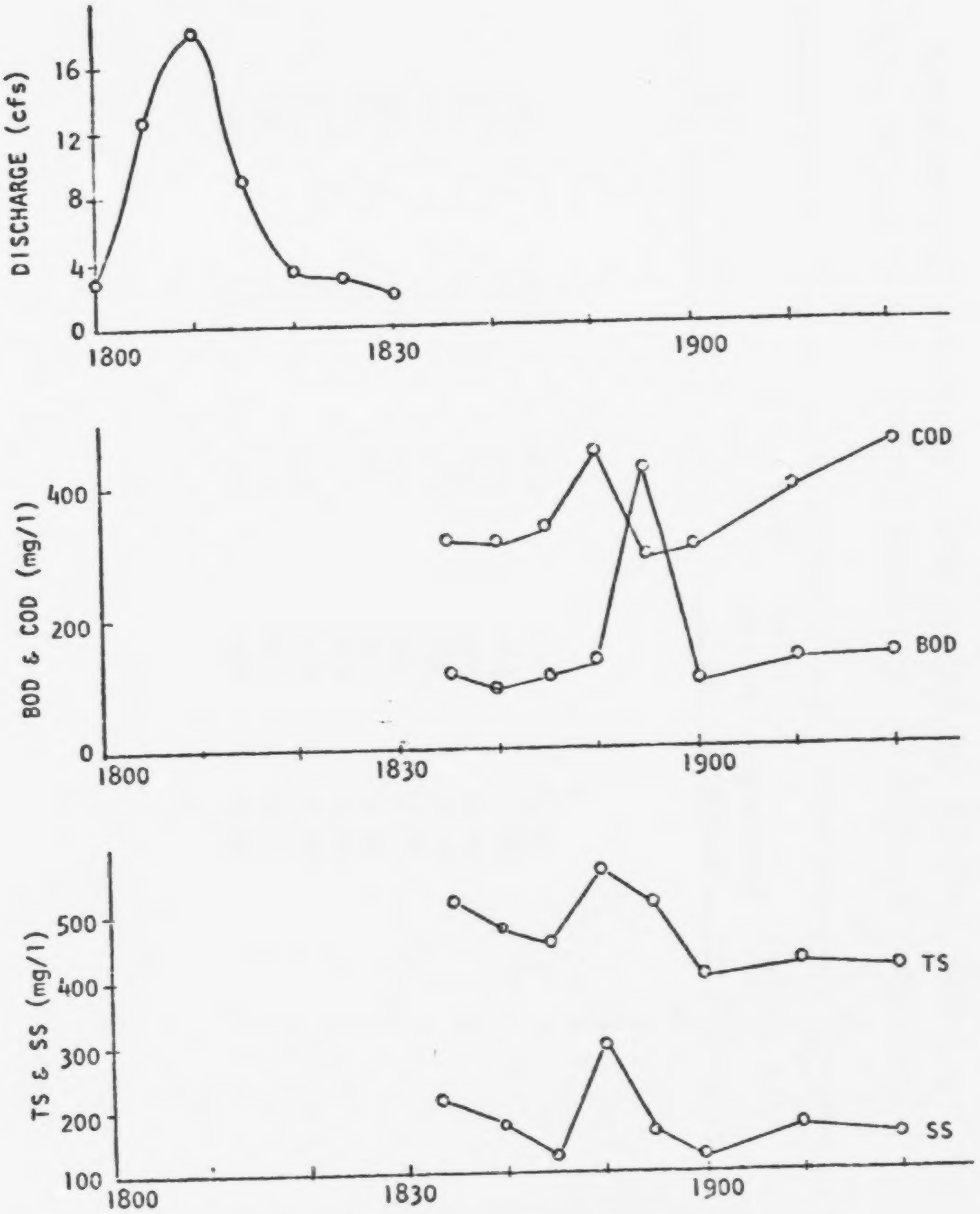


Figure 98. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 24, 1970
MONITORING STATION: BANK - 96" SEWER

Table 37 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: September 24, 1970 MONITORING STATION: Longview - 10' x 15' Sewer

Time	Intensity of Rain (in/hr)	Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	BOD	COD
	Woodward Gage					
1800	0.84					
1810	0.0					
1820	0.24					
1830	0.0					
1835		39.0	697	488	135	208
1840		100.0	691	428	130	208
1845		94.0	594	352	120	228
1850		84.0	584	264	115	152
1855		79.0	588	308	110	240
1900		78.0	510	178	80	128
1905		64.0	476	192	95	116
1910		40.0	499	266	115	108
1915		31.0	316	110	90	188
1920		39.0	488	235	105	220

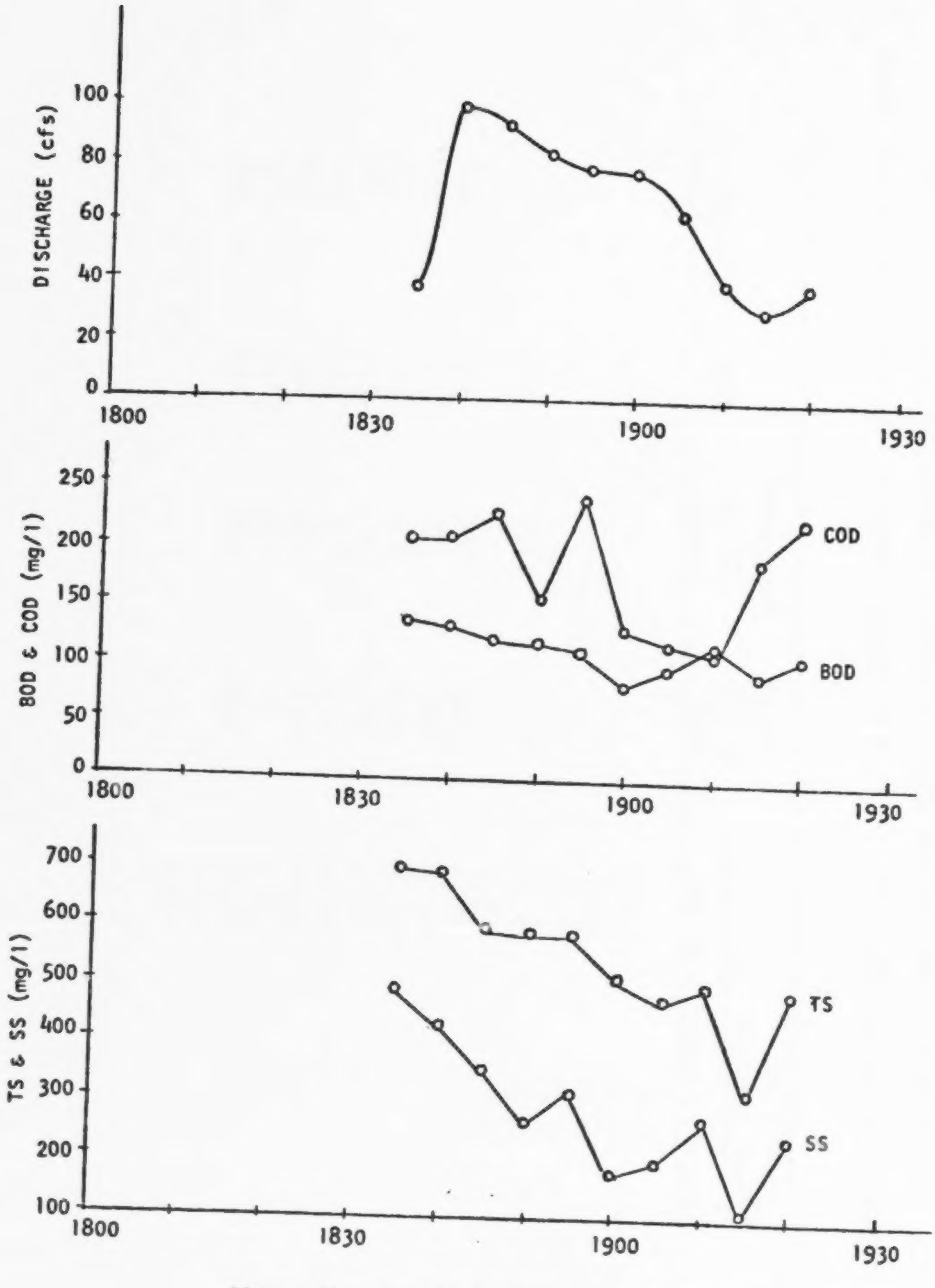


Figure 99. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 24, 1970
MONITORING STATION: LONGVIEW - 10' x 15' SEWER

Table 38 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: September 24, 1970 MONITORING STATION: Outlet - 10' x 15' Sewer

Time	Intensity of Rain (in/hr)	Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	BOD	COD
	Woodward Gage					
1800	0.80					
1807:30	1.32	10.0				
1815	0.12	30.0				
1822:30	0.24	53.0				
1828:30	0.36	90.0	3017	2356	415	828
1837:30	0.0	67.5				
1840:30		88.0	1946	1764	385	828
1848		107.0	985	536	625	860
1855:30		97.0	847	580	285	548
1903		95.0	614	264	220	484
1910:30		94.0	573	204	210	840
1918		93.0	621	436	420	1200
1925:30		92.5	431	272	210	600
1933		85.0	499	324	260	596
1945		22.0				
2000		18.0				

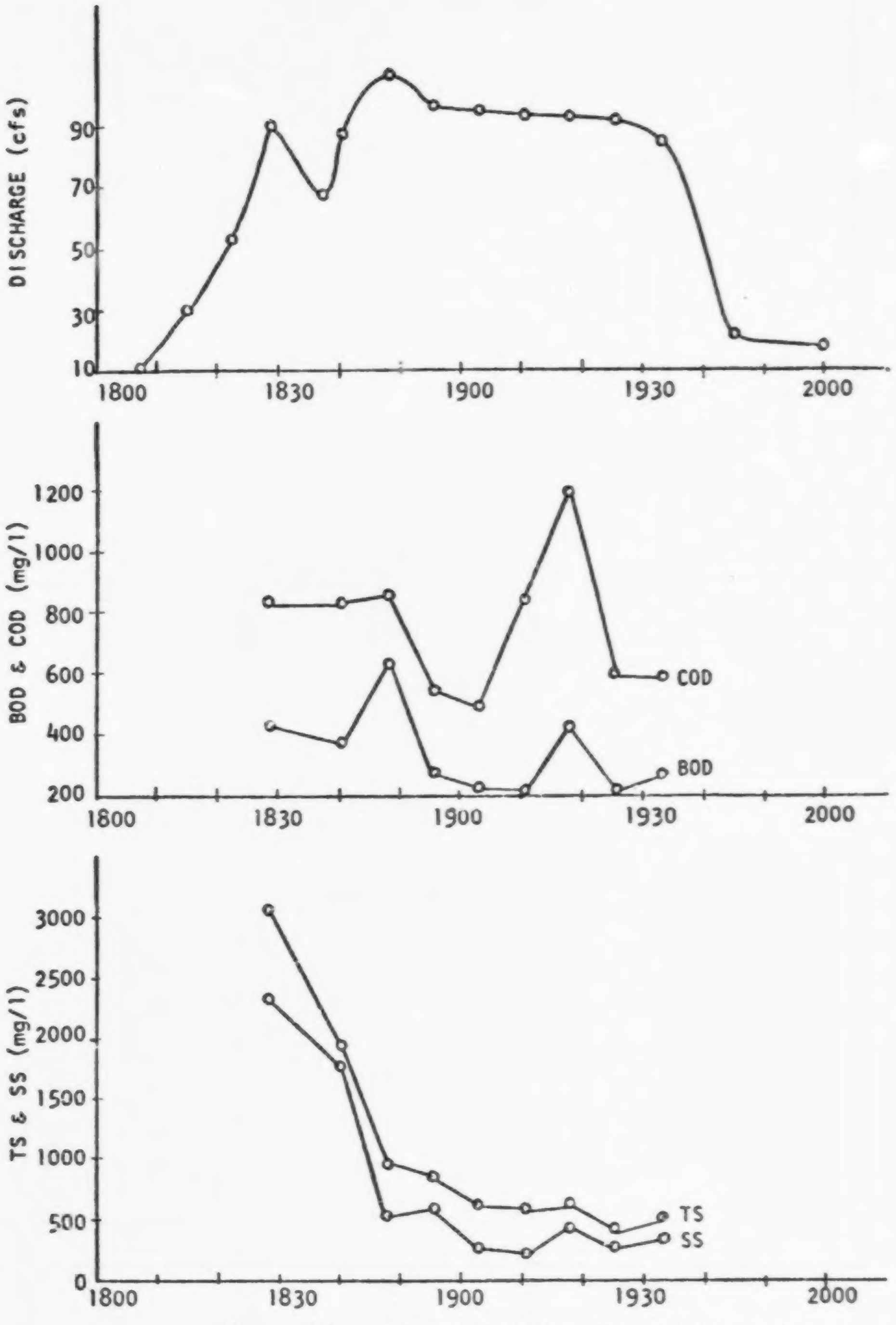


Figure 100. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 24, 1970
MONITORING STATION: OUTLET - 10' x 15' SEWER

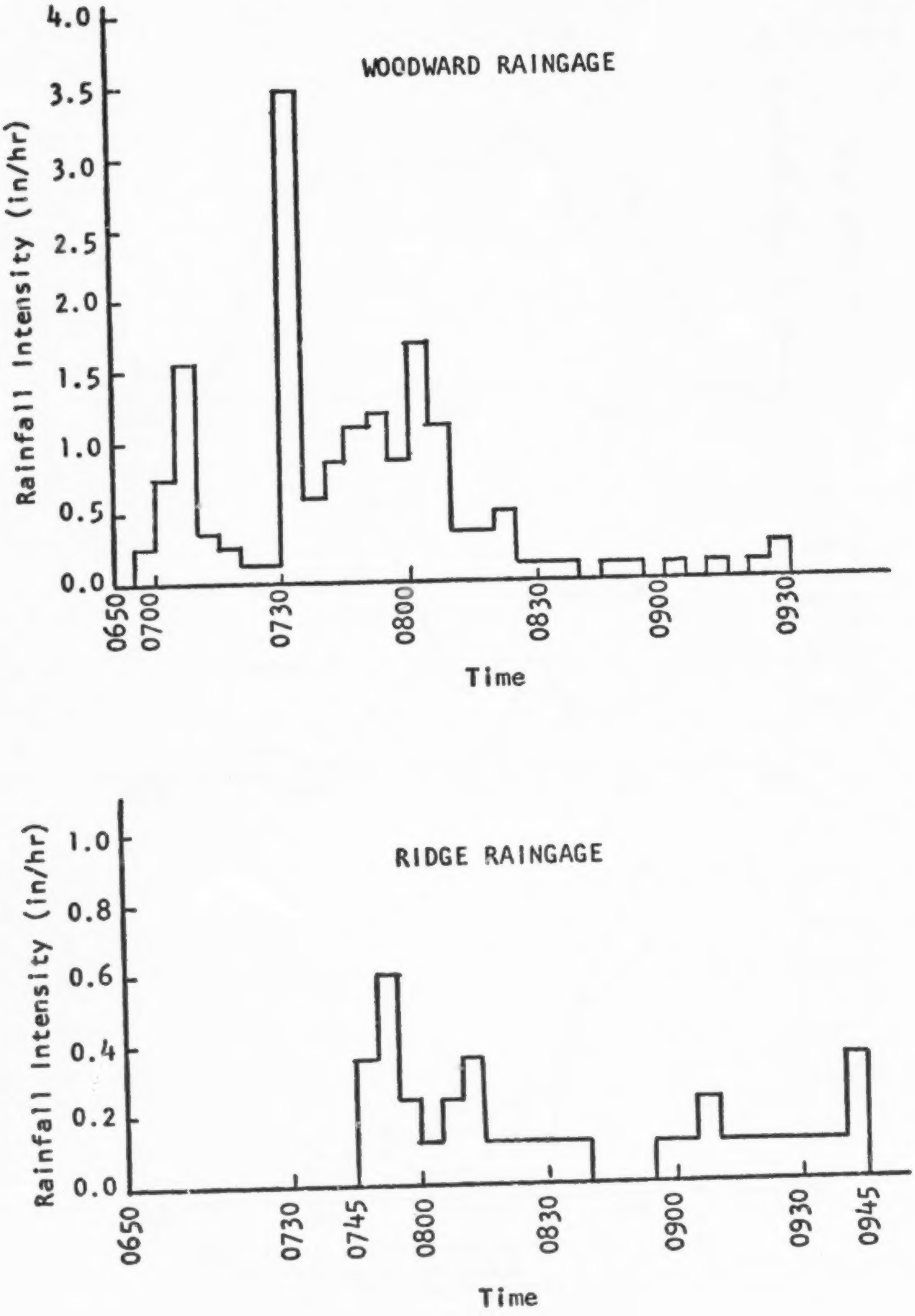


Figure 101. RAIN OF SEPTEMBER 25, 1970 HYETOGRAPHS

Table 39 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: September 25, 1970   MONITORING STATION: Bank - 54" Sewer

Time	Intensity of Rain (in/hr)		Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	BOD	COD
	Ridge Gage	Woodward Gage					
0650		0.00	1.0				
0705		1.56	21.5				
0720		0.12	4.50				
0730		3.48	51.5				
0740	0.00	0.84	3.0				
0800	0.12	1.68	8.0				
0835	0.12	0.12	2.0	410	196	-	16
0845	0.00	0.12		528	276	30	208
0855	0.12	0.00		416	136	45	240
0907	0.24	0.12		310	72	35	-
0920	0.12	0.12		310	114	25	188
0935	0.12	0.0		316	116	50	-
1000	0.00	0.0		514	228	30	212

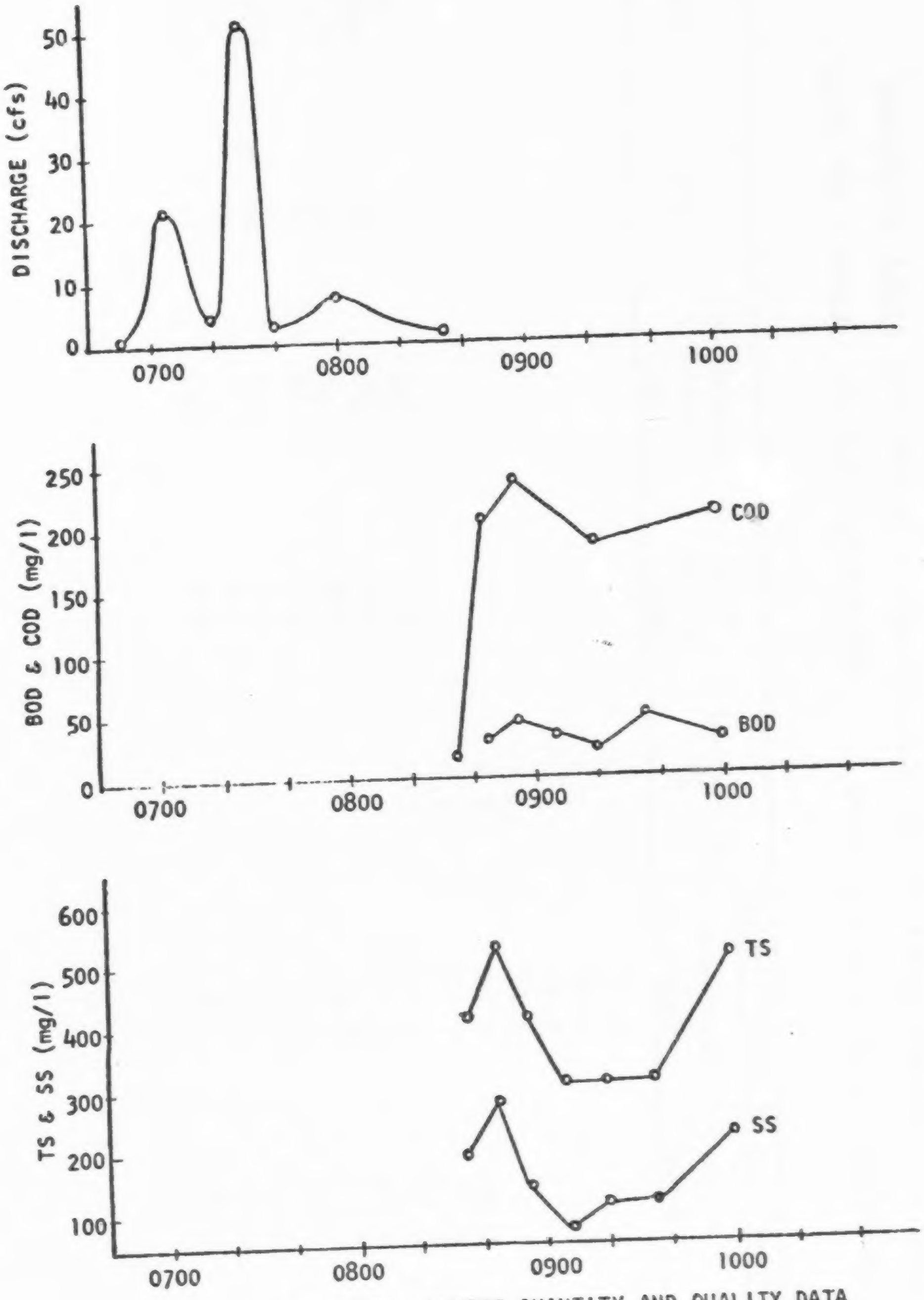


Figure 102. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 25, 1970
MONITORING STATION: BANK - 54" SEWER

Table 40 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: September 25, 1970 MONITORING STATION: Bank - 78" Sewer

Time	Intensity of Rain (in/hr)		Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	BOD	COD
	Ridge Gage	Woodward Gage					
0800	0.12	1.68	1.00				
0810	0.36	0.36	2.50				
0820	0.12	0.48	2.00				
0830	0.12	0.12	6.50				
0843	0.0	0.0	1.0	502	152	35	236
0850	0.0	0.12	1.5	-	-	45	308
0900	0.12	0.12	1.0	875	684	60	228
0910	0.12	0.12	1.0	472	168	60	104
0925	0.12	0.24		416	148	20	48
0950	0.0	0.0		318	64	30	232
1010	0.0	0.0		320	60	40	-

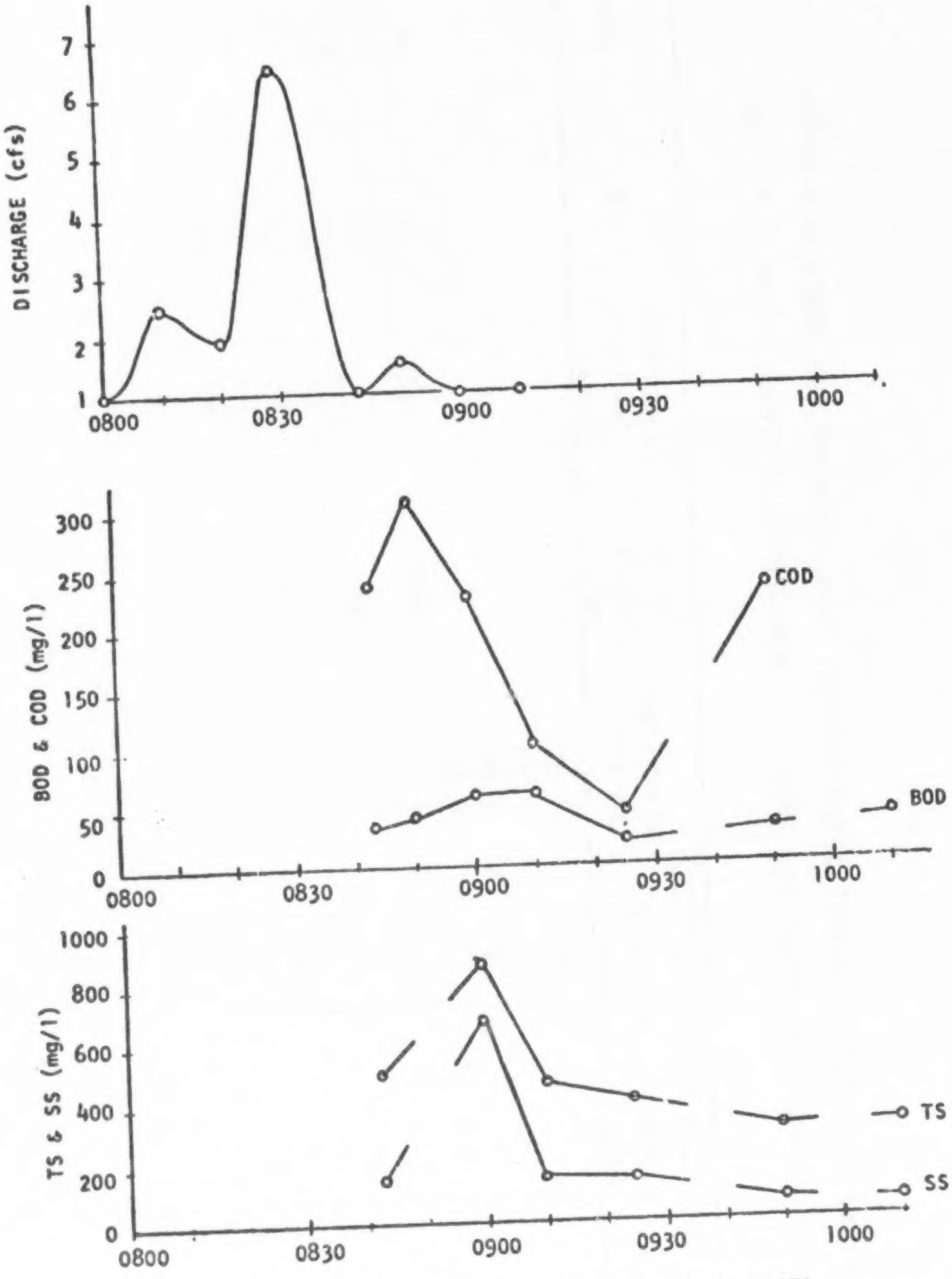


Figure 103. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 25, 1970
MONITORING STATION: BANK - 78" SEWER

Table 41 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: September 25, 1970 MONITORING STATION: Bank - 96" Sewer

Time	Intensity of Rain (in/hr) Ridge Gage	Intensity of Rain (in/hr) Woodward Gage	Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	BOD	COD
0820	0.12	0.48	3.0	-	-	15	220
0825	0.12	0.12	5.5	-	-	20	212
0830	0.12	0.12	7.5	276	88	40	496
0840	0.0	0.0	2.0	510	268	40	300
0852	0.0	0.12	1.2	-	-	45	-
0900	0.12	0.12		472	200	40	-
0915	0.12	0.12		413	124	60	308
0930	0.12	0.0		377	84	30	48
0945	0.0			-	-	35	64
1020				527	244	30	240
1035				358	66	20	56

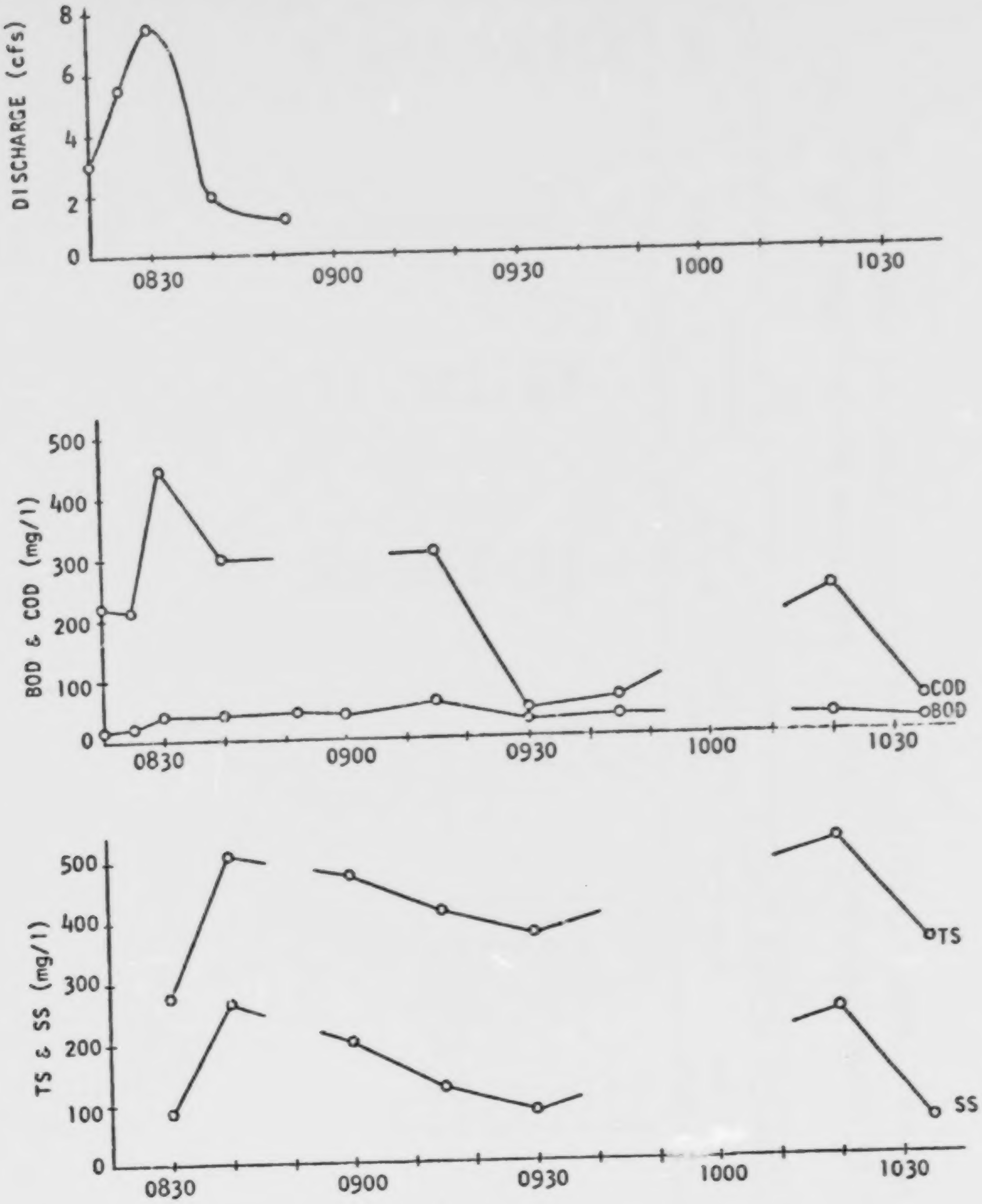


Figure 104. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 25, 1970
MONITORING STATION: BANK - 96" SEWER

Table 42 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: September 25, 1970 MONITORING STATION: Longview - 10' x 15' Sewer

Time	Intensity of Rain (in/hr)		Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	BOD	COD
	Ridge Gage	Woodward Gage					
0850	0.0	0.12	70	376	184	35	160
0855	0.12	0.0	74	410	176	35	172
0900	0.12	0.12	75	488	202	35	204
0910	0.24	0.12	80	471	198	40	204
0915	0.12	0.12	86	419	132	50	208
0920	0.12	0.12	90	427	154	55	216
0925	0.12	0.24	87	412	130	40	208
0930	0.12	0.0	66	398	198	50	224
0935	0.12		76	576	288	55	268
0940	0.36		70	447	182	45	216
0945	0.0		66	402	118	45	224
0950			76	399	108	45	208
1000			90	386	116	50	204
1010			70	323	84	45	204
1020			56	312	58	55	208
1030			43	329	74	60	220
1040			34	386	91	35	212
1050			28	375	88	60	240

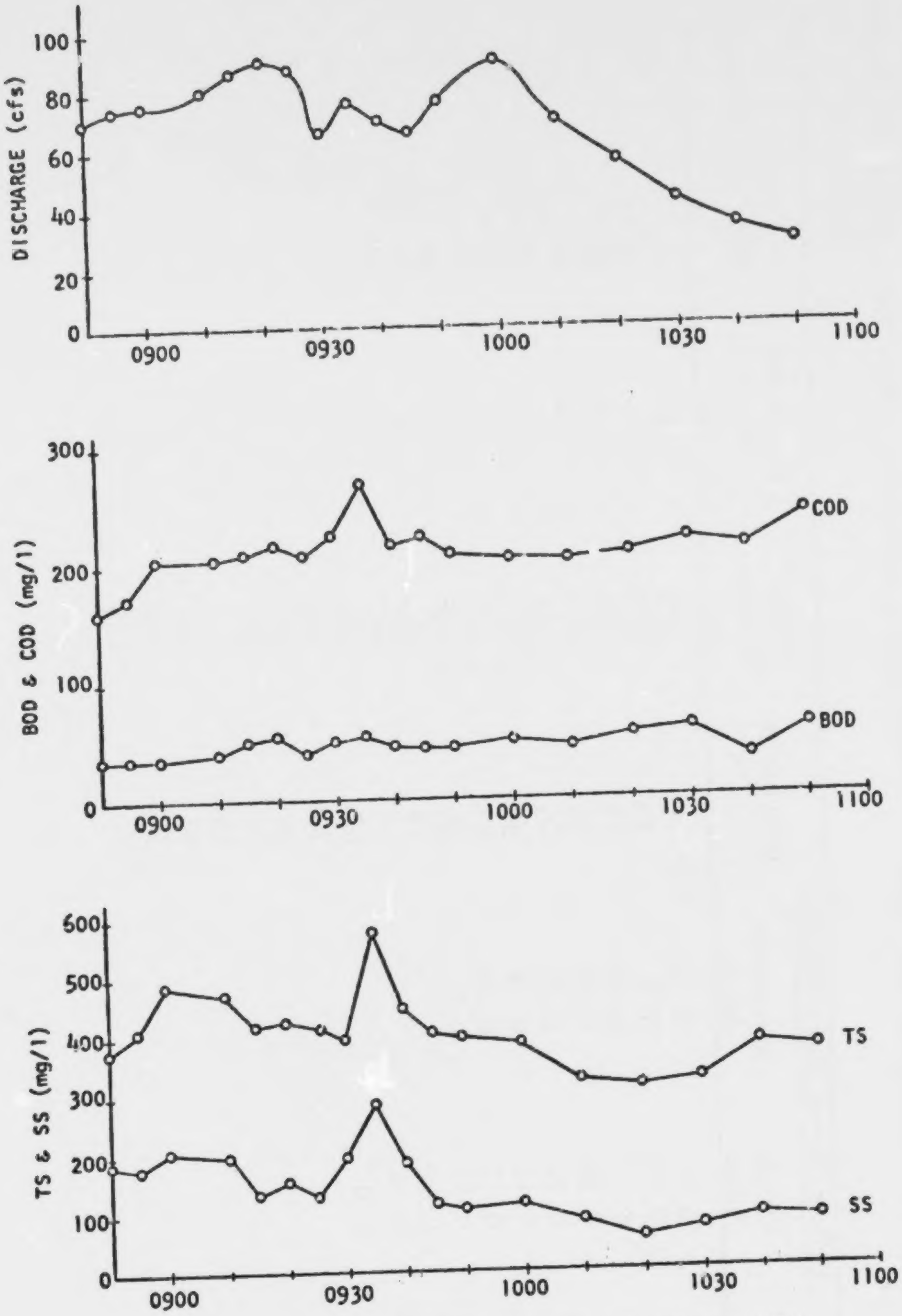


Figure 105. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 25, 1970
MONITORING STATION: LONGVIEW - 10' x 15' SEWER

Table 43 - RUNOFF QUANTITY AND QUALITY DATA - WOODY RUN SEWER WATERSHED

DATE OF STORM: September 25, 1970 MONITORING STATION: Outlet - 10' x 15' Sewer

Time	Intensity of Rain (in/hr)		Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	Volatile Suspended Solids mg/l	BOD	COD
	Ridge Gage	Woodward Gage						
0742	0.36	0.84	93.0	568	292	152	105	397
0749:30	0.36	1.08	100.0	748	416	203	85	432
0757	0.24	0.84	102.0	961	636	362	45	320
0804:30	0.12	1.68	101.0	3275	2676	1248	70	316
0812	0.36	0.36	97	1276	932	412	75	340
0819:30	0.12	0.36	95	997	748	316	85	500
0827	0.12	0.12	94	1033	702	366	85	344
0834:30	0.12	0.12	96	841	508	256	55	260
0842	0.0	0.0	95	684	396	162	55	248
0849:30	0.0	0.12	94	617	312	138	45	244
0857	0.12	0.0	94	699	408	278	60	244
0904:30	0.12	0.12	94	621	294	152	65	244
0912	0.12	0.12	95	672	368	134	70	268
0919:30	0.12	0.0	95	584	280	110	70	288
0927	0.12	0.24	96	685	392	182	65	276
0934:30	0.12	0.0	95	580	328	149	65	288
0942	0.36		95	572	308	133	90	300
0949:30	0.0		94	-	-	-	-	-
0957			94	-	-	-	85	316
1004:30			94	-	-	-	65	284
1012			94	778	428	202	75	300
1019:30			94	677	356	135	85	308
1027			93	482	160	89	185	304

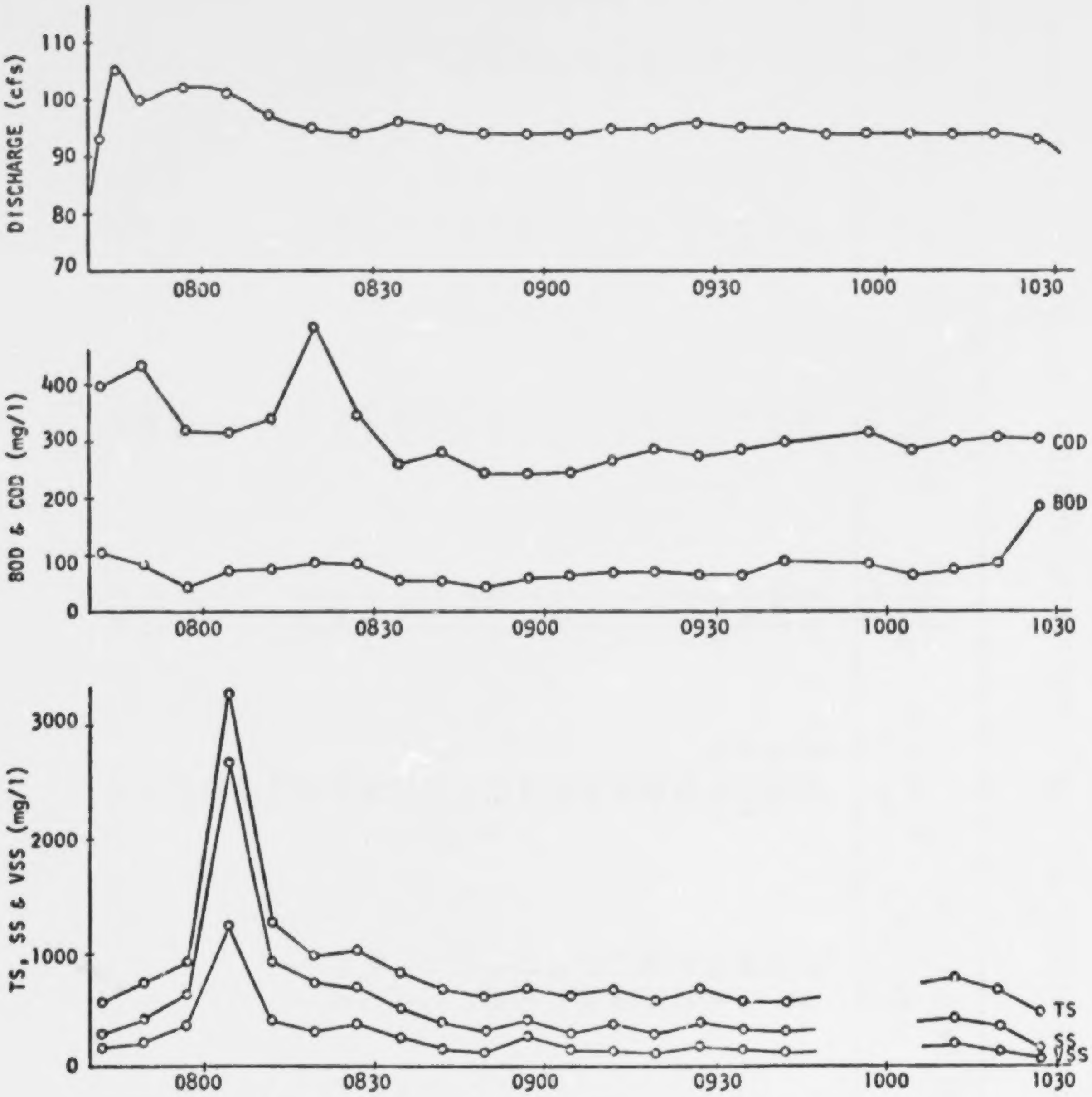


Figure 106. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 25, 1970
MONITORING STATION: OUTLET - 10' x 15' SEWER

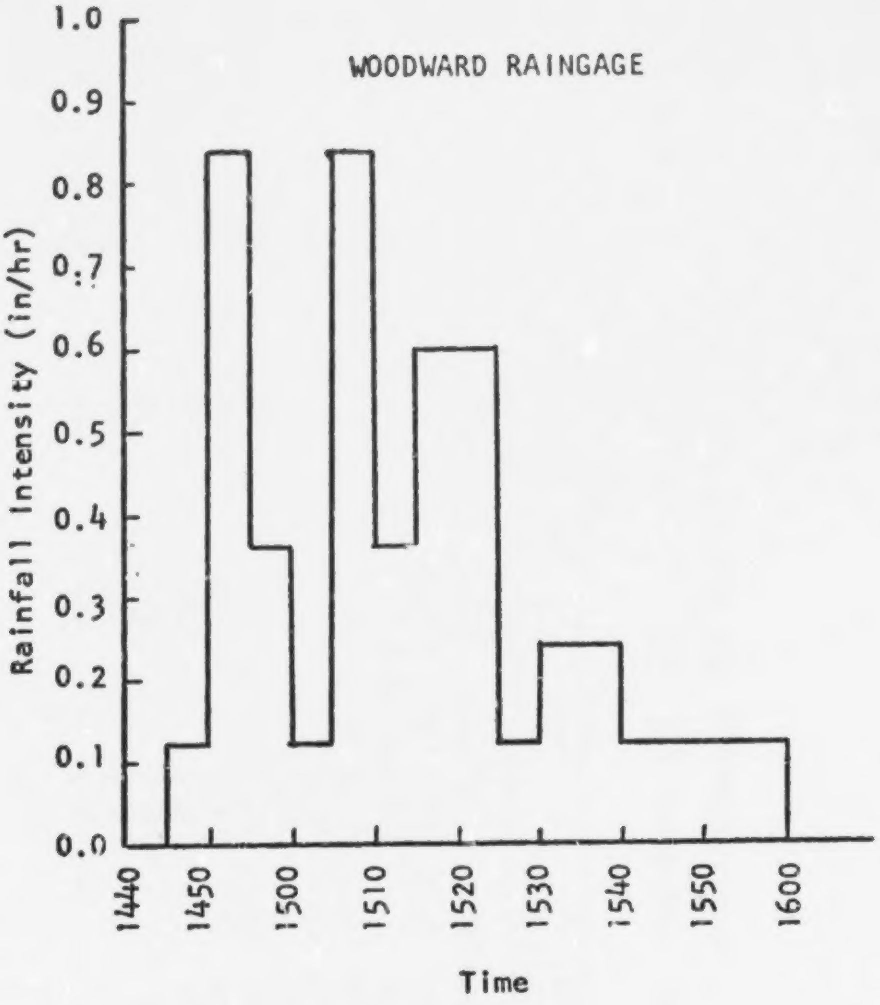


Figure 107. RAIN OF SEPTEMBER 26, 1970 HYETOGRAPH*

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*An additional 0.35 inches of rain fell in the next 11 hours

Table 44 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: September 26, 1970 MONITORING STATION: Bank - 54" Sewer

Time	Intensity of Rain (in/hr) Woodward Gage	Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	Volatile Suspended Solids mg/l	BOD	COD
1450	0.84	1.0					
1510	0.36	10.0					
1515	0.60	41.0					
1525	0.12	16.5					
1535	0.24	28					
1540	0.12	18.5	238	74	38	45	224
1550	0.12	11.0	189	57	-	55	220
1600	0.12	8.5	482	190	-	55	224
1610	0.0	6.5	331	126	-	45	216
1620		4.5	303	127	64	55	224
1635		2.2	317	140	78	65	260

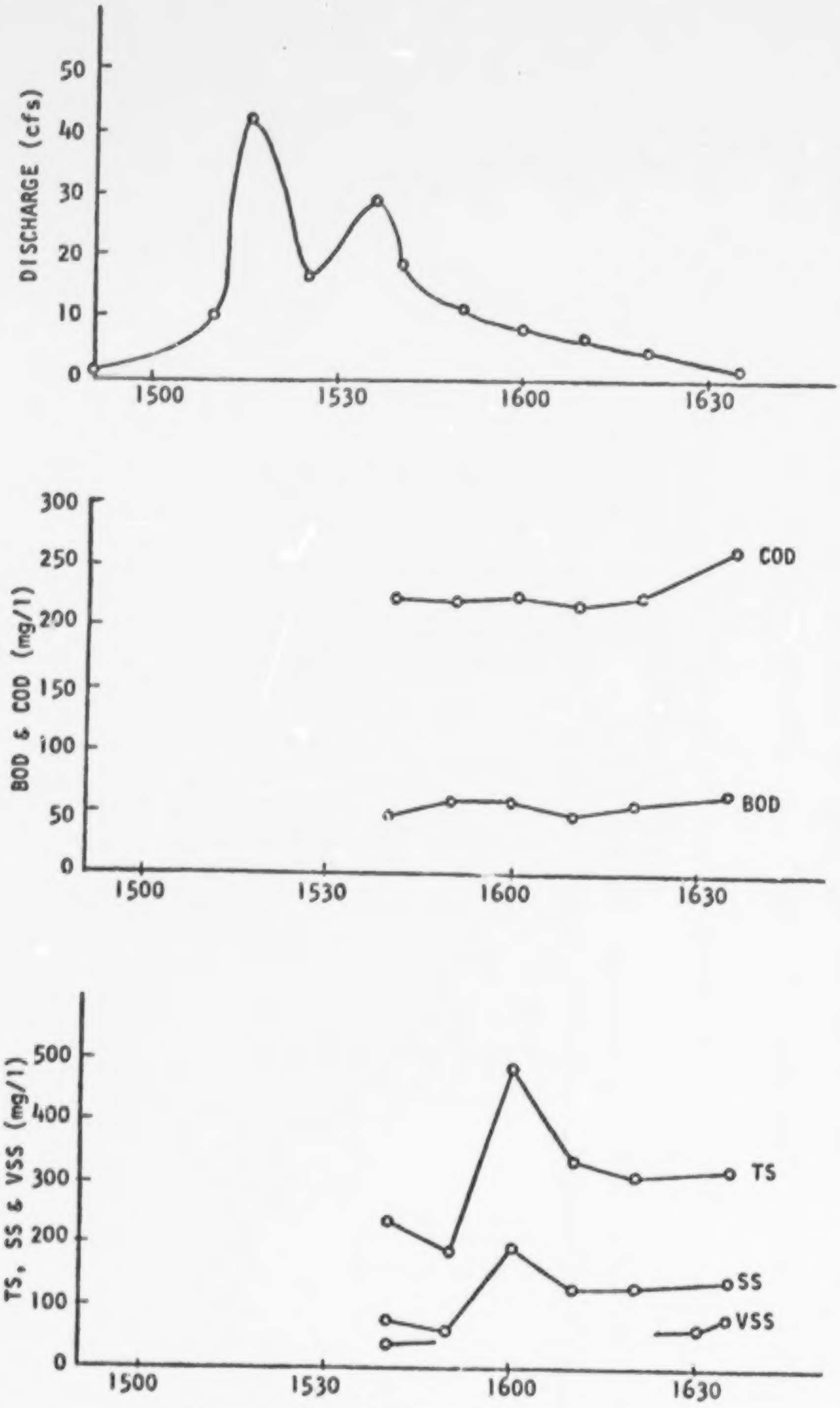


Figure 108. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 26, 1970
MONITORING STATION: BANK - 54" SEWER

Table 45 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: September 26, 1970 MONITORING STATION: Bank - 78" Sewer

Time	Intensity of Rain (in/hr)	Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	Volatile Suspended Solids mg/l	BOD	COD
	Woodward Gage						
1520	0.60	4.50					
1530	0.24	20.0					
1535	0.24	25.0	259	99	56	65	272
1545	0.12	44.5	-	-	-	25	-
1555	0.12	12.0	306	101	-	40	220
1605	0.0	8.0	376	264	-	25	155
1615		10.0	345	142	77	35	276
1630		4.0	374	184	96	65	260
1645		4.0	214	63	42	65	260
1700		2.5					

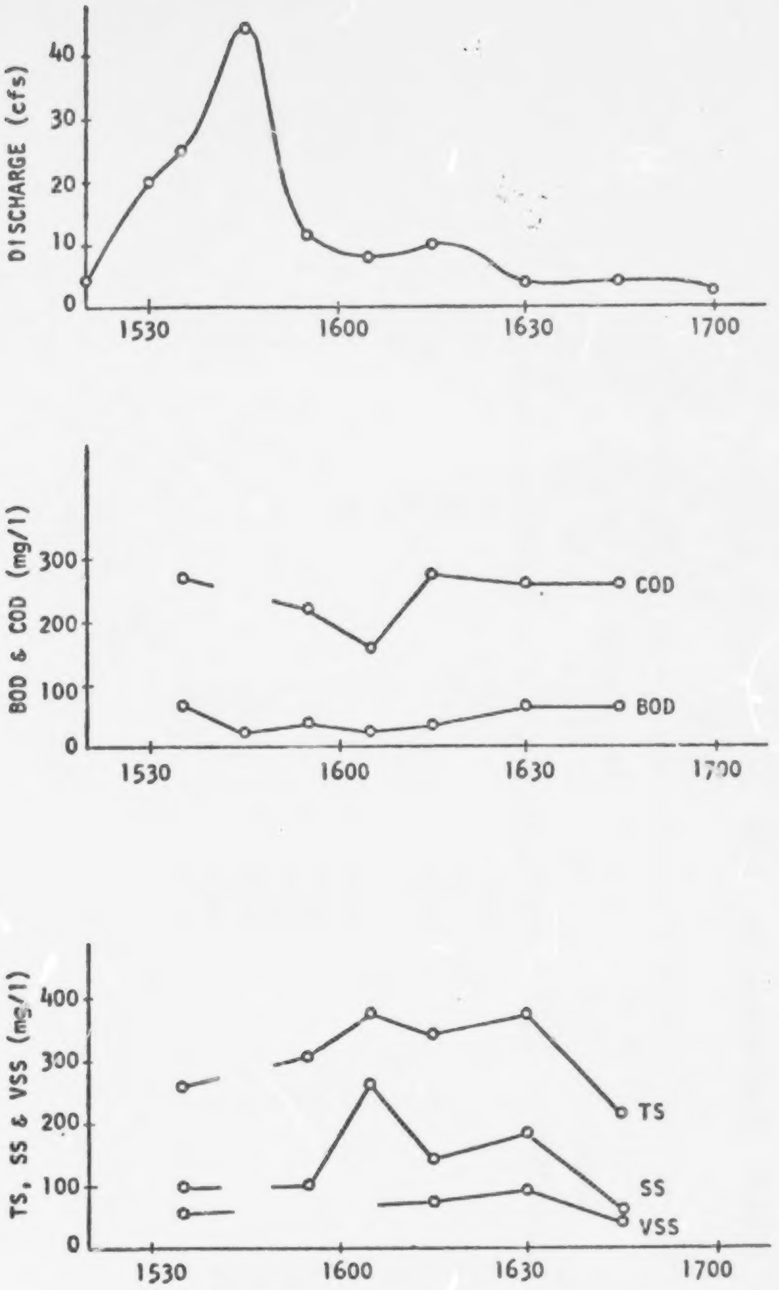


Figure 109. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 26, 1970
MONITORING STATION: BANK - 78" SEWER

Table 46 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: September 26, 1970 MONITORING STATION: Longview - 9' x 11' Sewer

Time	Intensity of Rain (in/hr) Woodward Gage	Total Solids mg/l	Suspended Solids mg/l	Volatile Suspended Solids mg/l	BOD	COD
1600	rain has stopped	347	198	48	115	-
1605		521	392	184	85	-
1610		846	492	276	105	-
1615		707	406	180	105	368
1620		739	434	174	105	368
1625		686	470	-	110	400
1630		317	162	-	105	420
1635		724	484	-	115	392
1645		664	382	-	110	380
1655		627	354	-	125	392

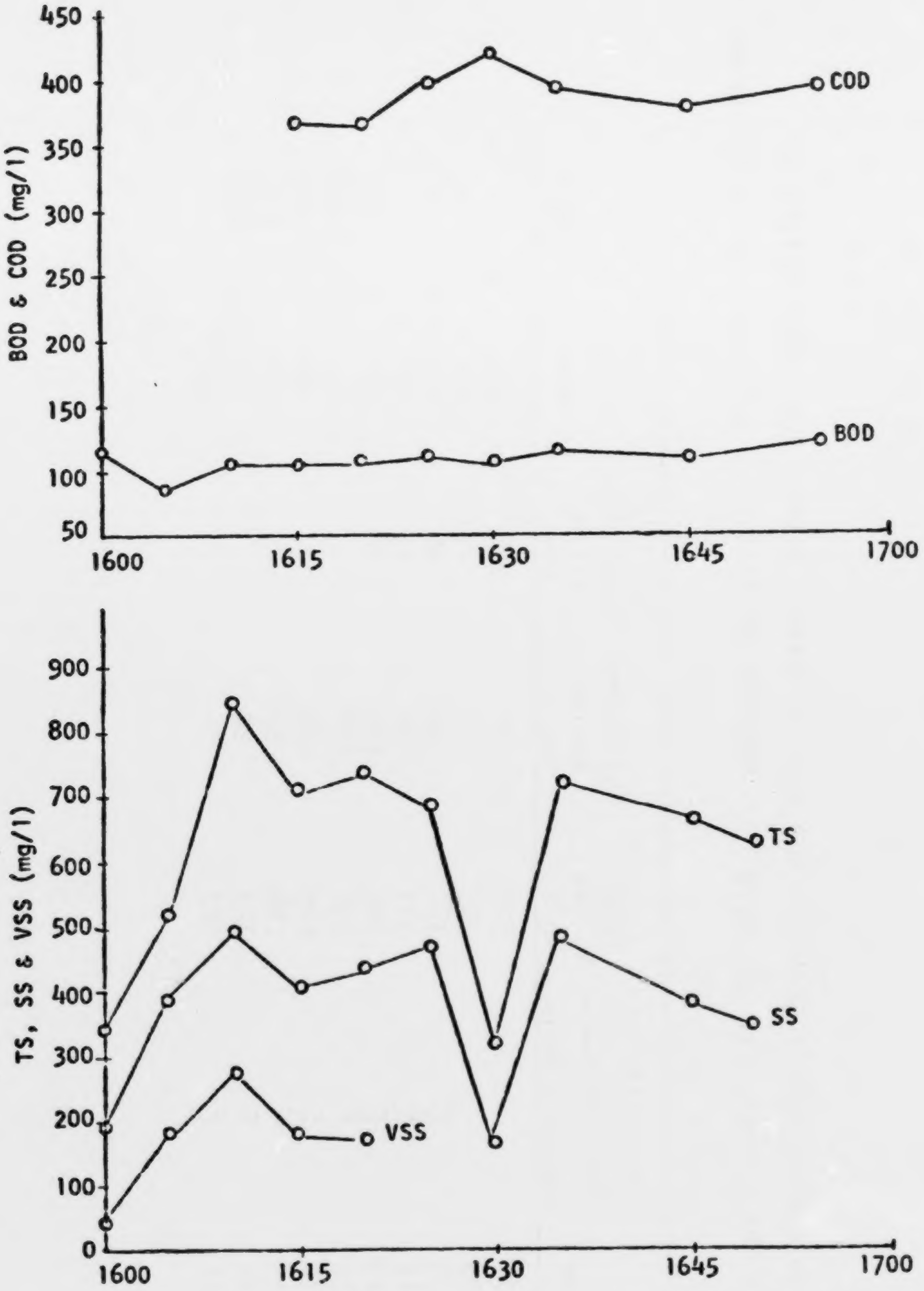


Figure 110. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 26, 1970
MONITORING STATION: LONGVIEW - 9' x 11' SEWER

Table 47 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: September 26, 1970 MONITORING STATION: Longview - 9' x 12' Sewer

Time	Intensity of Rain (in/hr) Woodward Gage	Total Solids mg/l	Suspended Solids mg/l	Volatile Suspended Solids mg/l	BOD	COD
1600	rain has stopped	575	430	228	55	-
1605		313	144	77	40	-
1610		371	154	96	35	-
1615		311	126	52	40	-
1620		286	130	44	35	244
1625		243	108	-	45	260
1630		609	406	-	45	252
1635		188	86	-	35	236
1645		349	62	-	25	228
1655		382	76	-	40	-

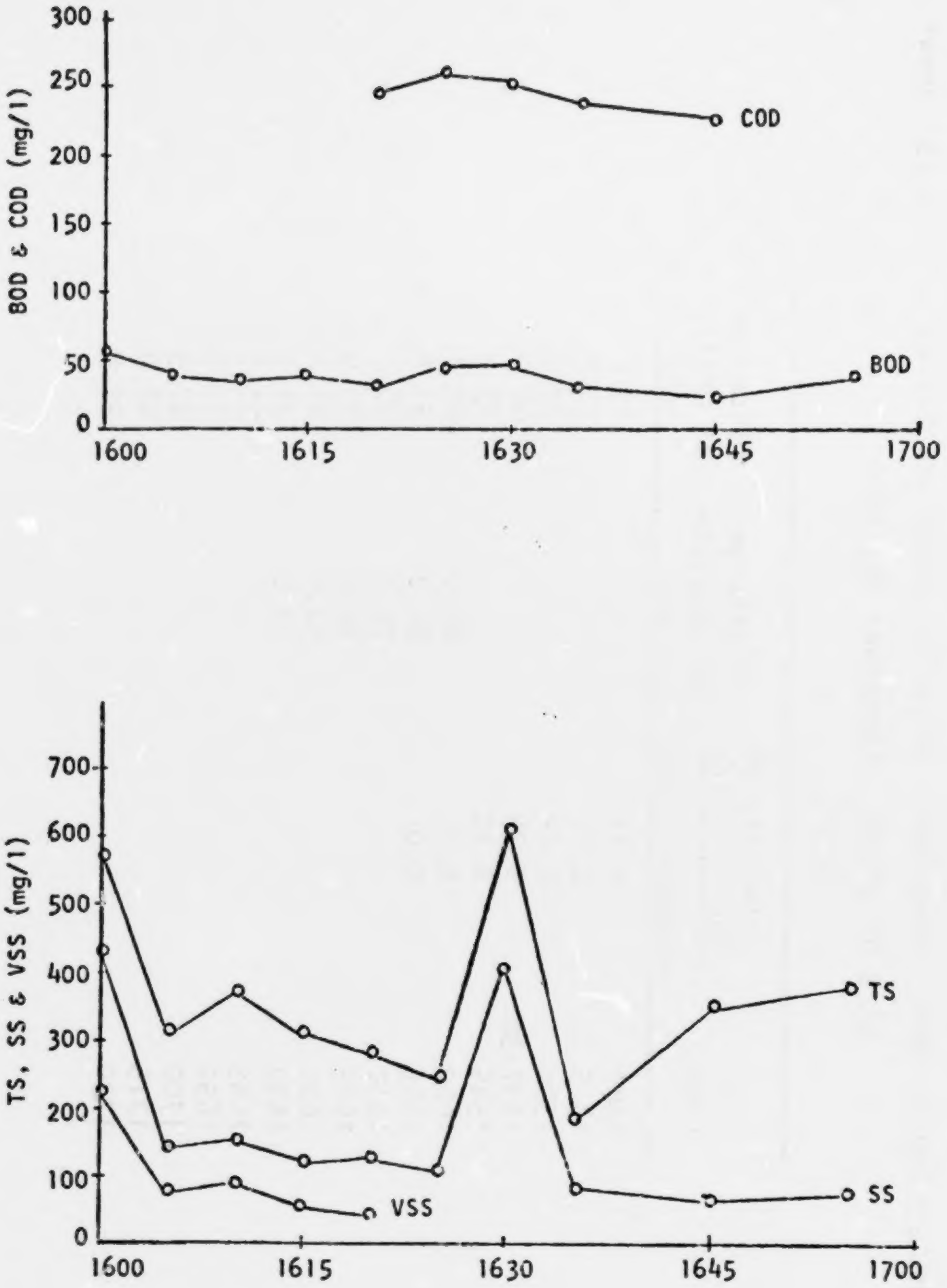


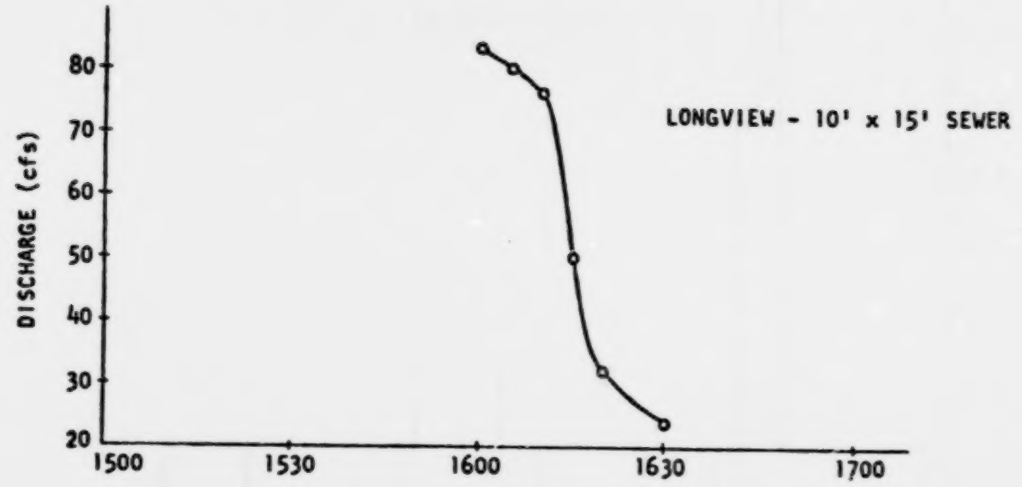
Figure 111. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 26, 1970
MONITORING STATION: LONGVIEW - 9' x 12' SEWER

Table 48 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DISCHARGE MEASUREMENTS FROM: Longview - 10' x 15' Sewer and Outlet - 10' x 15' Sewer

DATE OF STORM: September 26, 1970

Time	Intensity of Rain (in/hr) Woodward Gage	Discharge at Longview cfs	Discharge at Outlet cfs
1500	0.12		10.0
1515	0.60		35.0
1530	0.24		93.0
1537:30	0.24		68.0
1545	0.12		133.4
1600	0.0	83.0	100.0
1605	0.0	80.0	98.5
1610		76.0	96.0
1615		50.0	95.0
1620		32.0	95.0
1630		24.0	93.0
1645			92.5
1650			75.0
1700			87.5
1715			42.0
1730			22.0



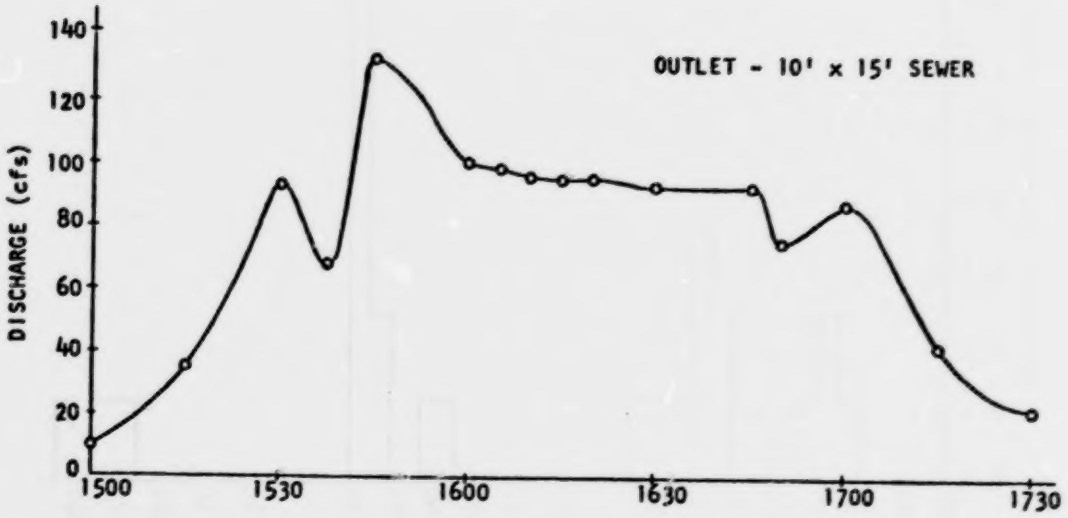


Figure 112. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: SEPTEMBER 28, 1970

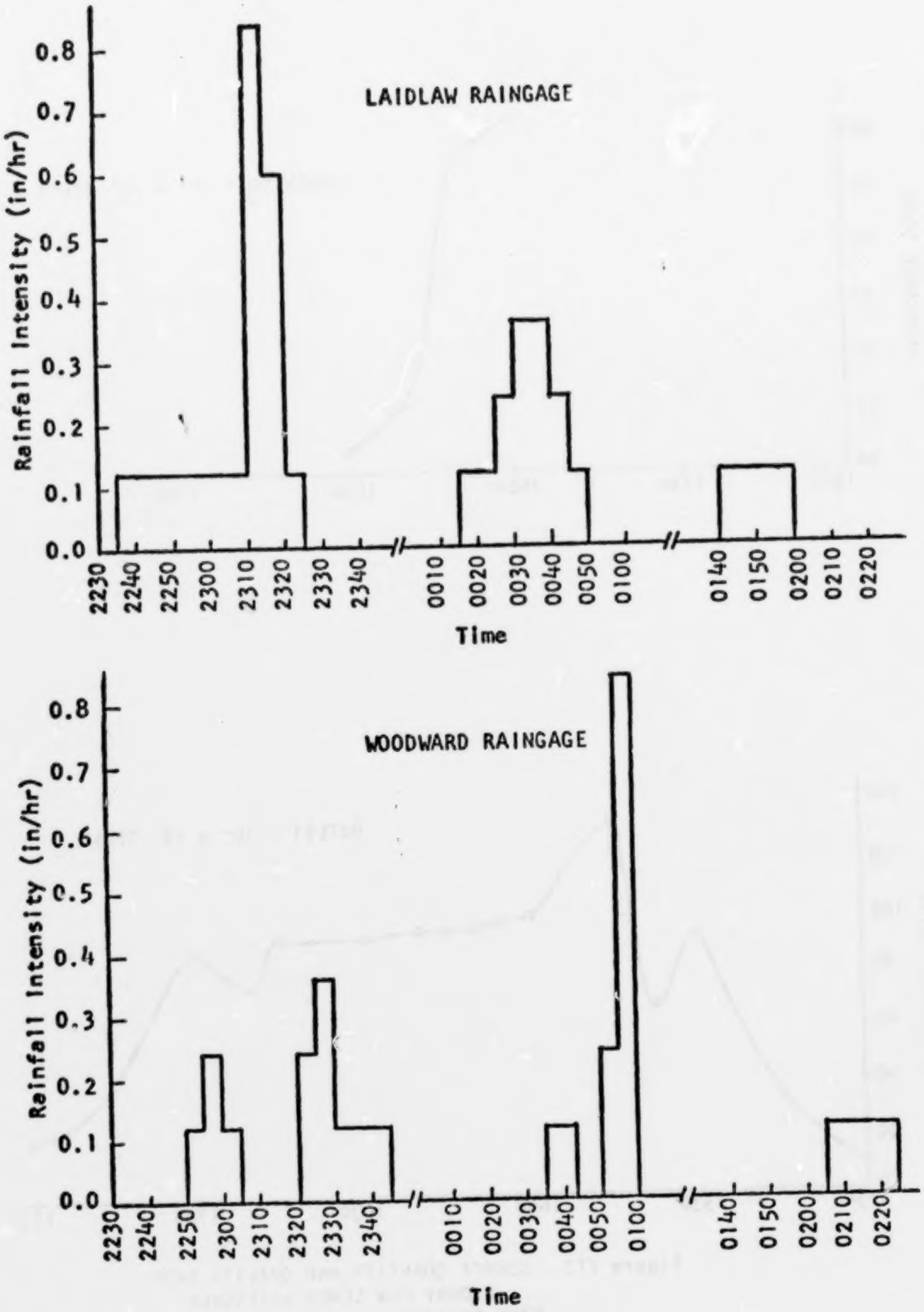


Figure 113. RAIN OF OCTOBER 9 & 10, 1970
HYETOGRAPHS

Table 49 . RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: October 9 & 10, 1970 MONITORING STATION: Bank - 54" Sewer

Time	Intensity of Rain (in/hr)		Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	Volatile Suspended Solids mg/l	BOD	COD	Chloride Concentration mg/l
	Laidlaw Gage	Woodward Gage							
2322	0.12	0.24	16.5						
2330	0.00	0.12	6.0						
2350	0.00	0.00		286	168	102	240	640	75
2400	0.00	0.00		264	162	100	185	408	25
0010	0.00	0.00		248	150	94	150	292	8
0025	0.24	0.00		222	132	86	82	292	33
0030	0.36	0.00	2.5						
0045	0.12	0.00	7.8						
0100	0.00	0.84	4.0	210	126	80	76	260	25
0108	0.00	0.00		194	121	74	72	236	17
0115	0.00	0.00		188	142	56	53	252	8
0125	0.00	0.00		176	106	58	60	216	8
0140	0.12	0.00		161	94	44	50	212	8

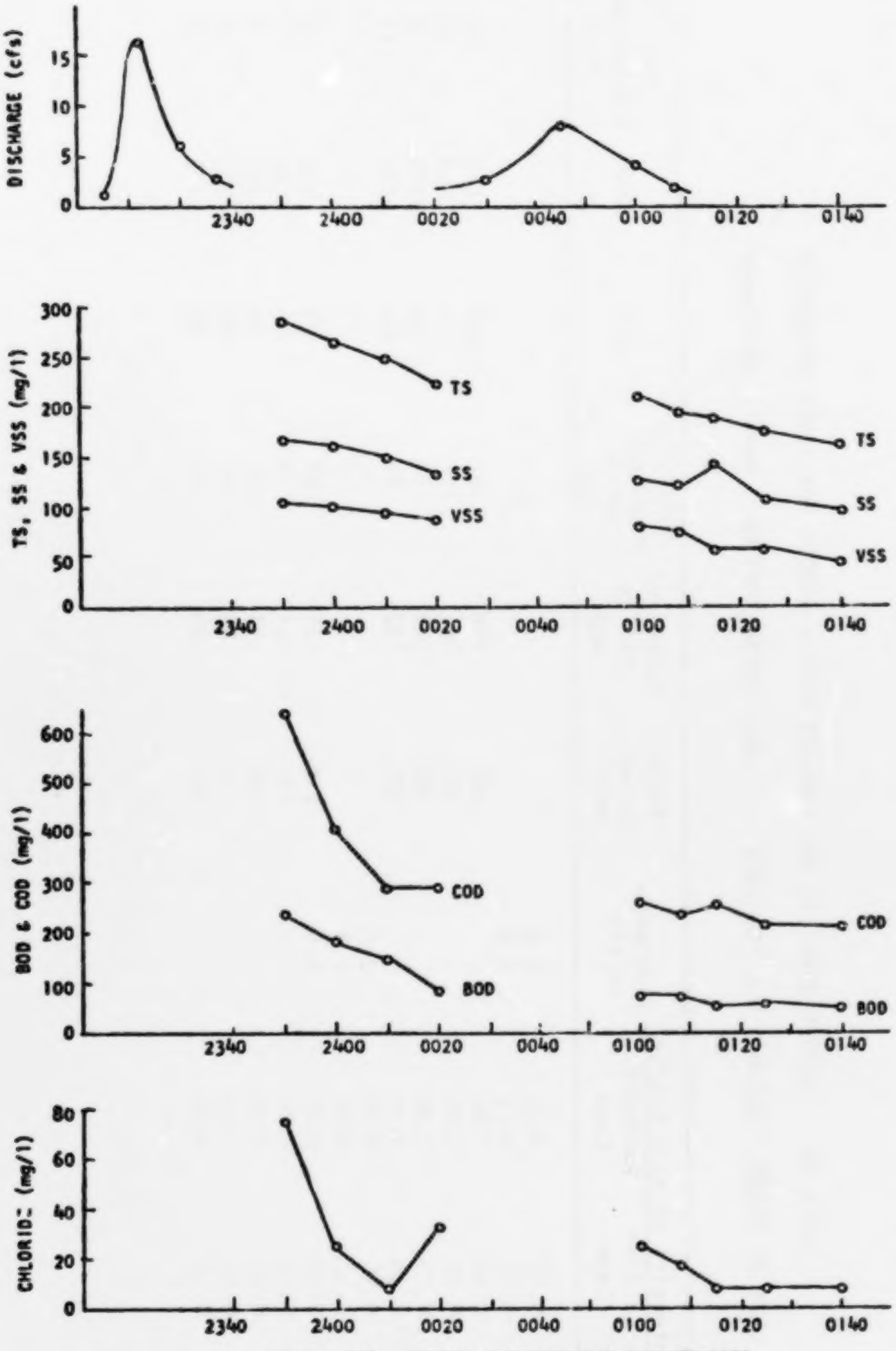


Figure 114. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: OCTOBER 9 & 10, 1970
MONITORING STATION: BANK - 54" SEWER

Table 50 . RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: October 9 & 10, 1970 MONITORING STATION: Bank - 78" Sewer

Time	Intensity of Rain (in/hr) Laidlaw Gage	Intensity of Rain (in/hr) Woodward Gage	Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	BOD	COD	Chloride Concentration mg/l
2330	0.00	0.12	15.0					
2338	0.00	0.12	35.0					
2345	0.00	0.00	6.0					
2355	0.00	0.00	4.0	246	178	115	424	83
0005	0.00	0.00	3.0	228	162	153	340	117
0015	0.12	0.00		214	154	102	248	42
0030	0.36	0.00		189	129	90	236	75
0052	0.00	0.24	7.0					
0103	0.00	0.00	14.0	176	122	53	128	16
0110	0.00	0.00	8.0	182	146	63	124	4
0120	0.00	0.00	2.0	148	131	60	120	8
0130	0.00	0.00	2.0	142	118	72	156	25
0145	0.12	0.00		136	104	53	156	50
0200	0.00	0.00	3.5					
0215		0.12	12.0					
0230		0.00	3.0					

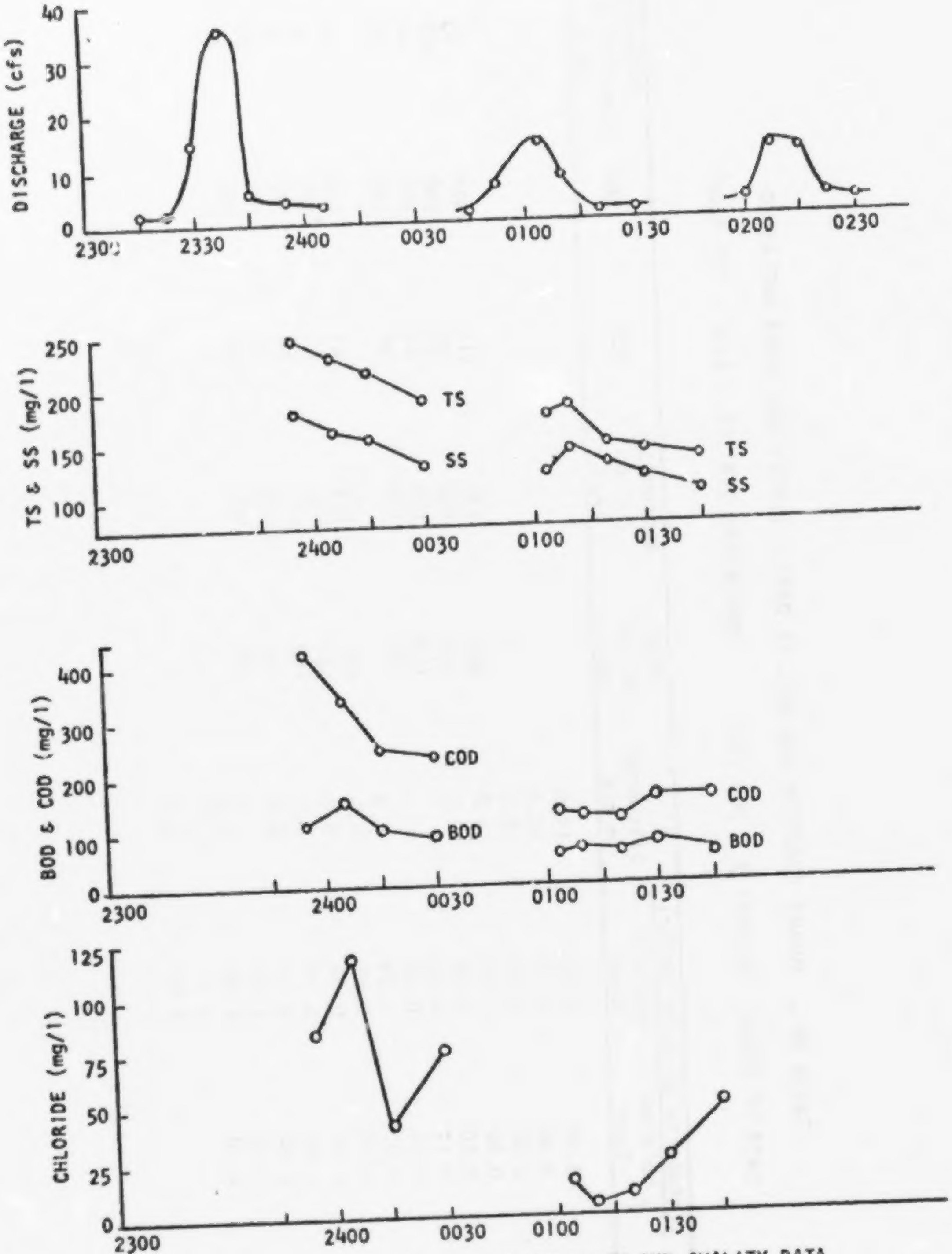


Figure 115. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: OCTOBER 9 & 10, 1970
MONITORING STATION: BANK - 78" SEWER

Table 51 . RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: October 9 & 10, 1970 MONITORING STATION: Longview - 11' x 9' Sewer

Time	Intensity of Rain (in/hr)		Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	Volatile Suspended Solids mg/l	BOD	COD	Chloride Concentration mg/l
	Laidlaw Gage	Woodward Gage							
2330	0.00	0.12	5.5						
2345	0.00	0.00	71.0						
2400	0.00	0.00	39.0						
0010	0.00	0.00	22.0	429	288	142	87	980	330
0015	0.12	0.00	14.0	416	276	134	94	896	110
0020	0.12	0.00	11.5	384	242	128	102	760	110
0025	0.24	0.00	8.5	356	224	116	88	546	70
0030	0.36	0.00	6.0	328	206	112	144	496	100
0100	0.00	0.00	18.0						
0107	0.00	0.00	29.0						
0115	0.00	0.00	25.0	302	198	98	110	546	200
0125	0.00	0.00	20.5	289	184	86	82	360	150
0200	0.00	0.00	5.0						
0230			15.3						
0300			6.0						

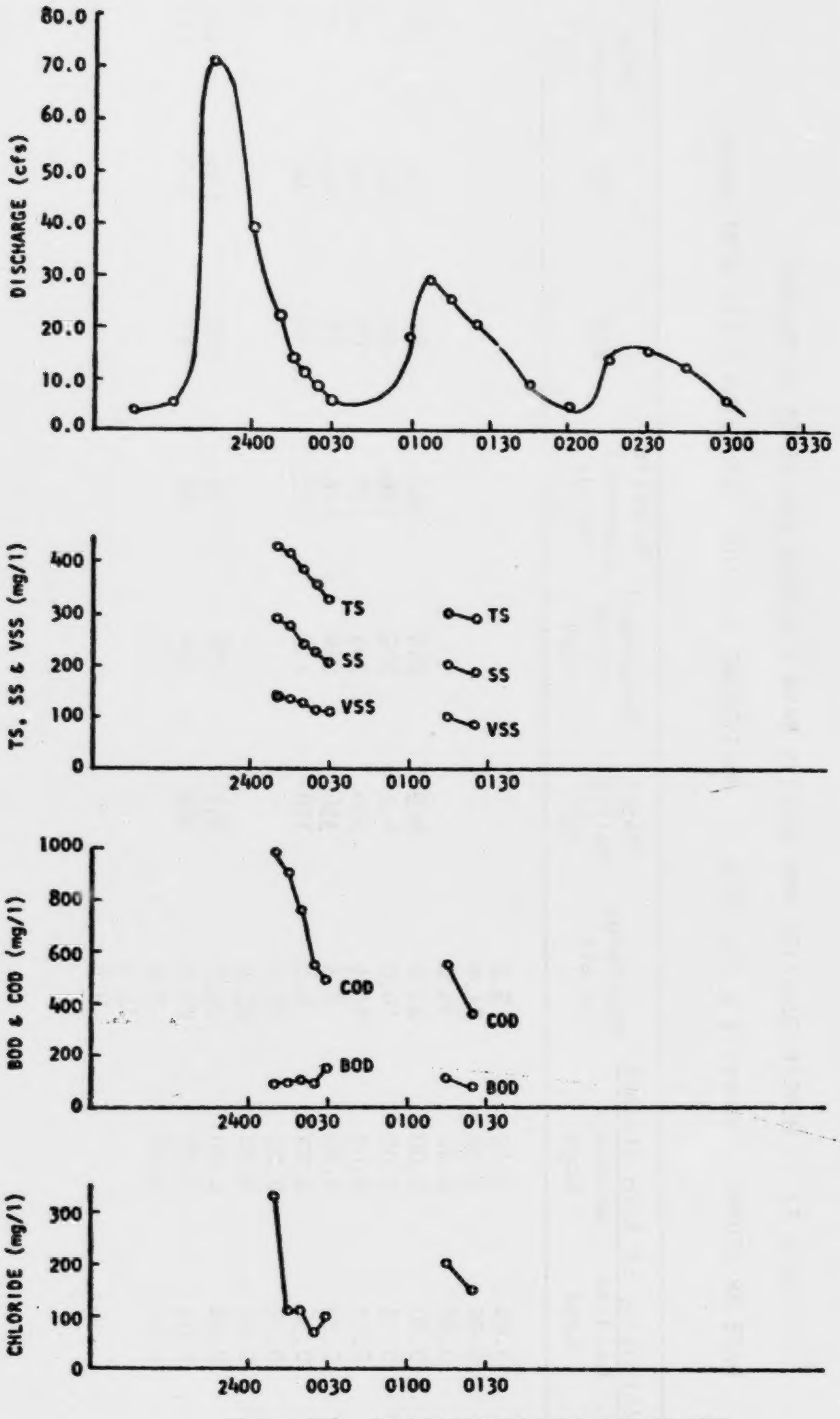


Figure 116. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: OCTOBER 9 & 10, 1970
MONITORING STATION: LONGVIEW - 11' x 9' SEWER

Table 52 . RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: October 9 & 10, 1970 MONITORING STATION: Longview - 12' x 9' Sewer

Time	Intensity of Rain (in/hr)		Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	BOD	COD	Chloride Concentration mg/l
	Laidlaw Gage	Woodward Gage						
2330	0.00	0.12	47.0					
2338	0.00	0.12	110.0					
0010	0.00	0.00	19.5	606	324	208	408	8
0015	0.12	0.00	14.5	588	306	83	348	1
0025	0.24	0.00	7.3	529	288	116	328	1
0030	0.36	0.00	6.0	428	274	142	316	8
0050	0.00	0.24	8.5	406	262	43	268	16
0110	0.00	0.00	43.5	442	241	60	252	10
0115	0.00	0.00	31.0	400	208	73	244	16
0125	0.00	0.00	20.0	392	192	60	236	3
0135	0.00	0.00	12.0	384	186	53	212	1
0200	0.00	0.00	5.5					
0230		0.00	23.0					
0300			7.0					

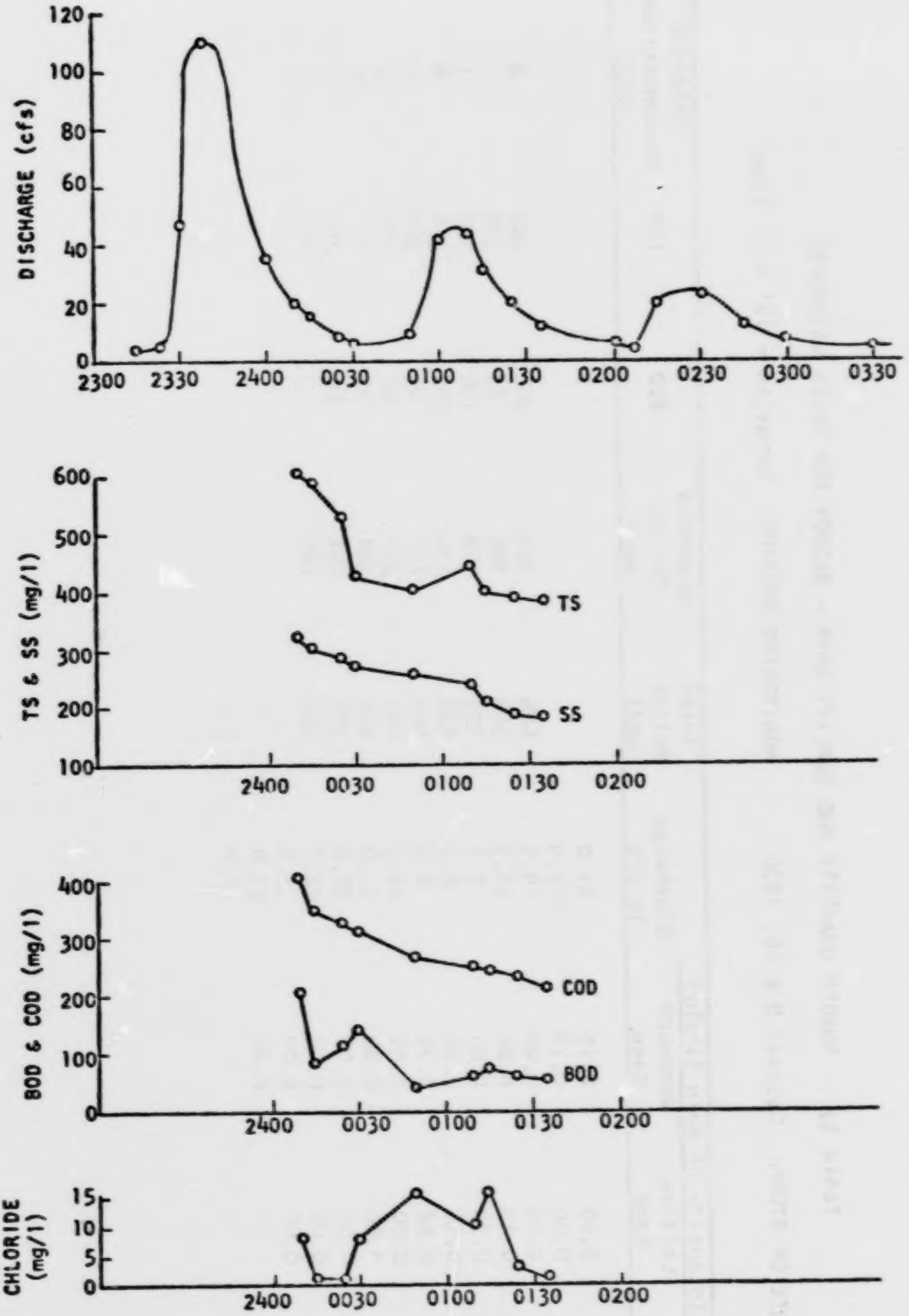


Figure 117. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: OCTOBER 9 & 10, 1970
MONITORING STATION: LONGVIEW - 12' x 9' SEWER

Table 53 . RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: October 9 & 10, 1970 MONITORING STATION: Outlet - 10' x 15' Sewer

Time	Intensity of Rain (in/hr) Laidlaw Gage	Intensity of Rain (in/hr) Woodward Gage	Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	BOD	COD	Chloride Concentration mg/l
2335	0.00	0.12	17.0					
2352:30	0.00	0.00	75.0	628	464	67	640	80
2400	0.00	0.00	56.0					
0022:30	0.12	0.00	56.0					
0045	0.12	0.12	16.0					
0107:30	0.00	0.00	44.0					
0115	0.00	0.00	30.0					
0122:30	0.00	0.00	75.0					
0145	0.12	0.00	49.5					
0215	0.00	0.12	12.0					
0240		0.00	58.0	640	428	230	540	50
0245			56.0	606	386	162	440	30
0255			46.0	584	324	118	308	10
0300			36.5	520	312	53	300	50
0310			16.5	504	406	67	296	100
0330			12.0					

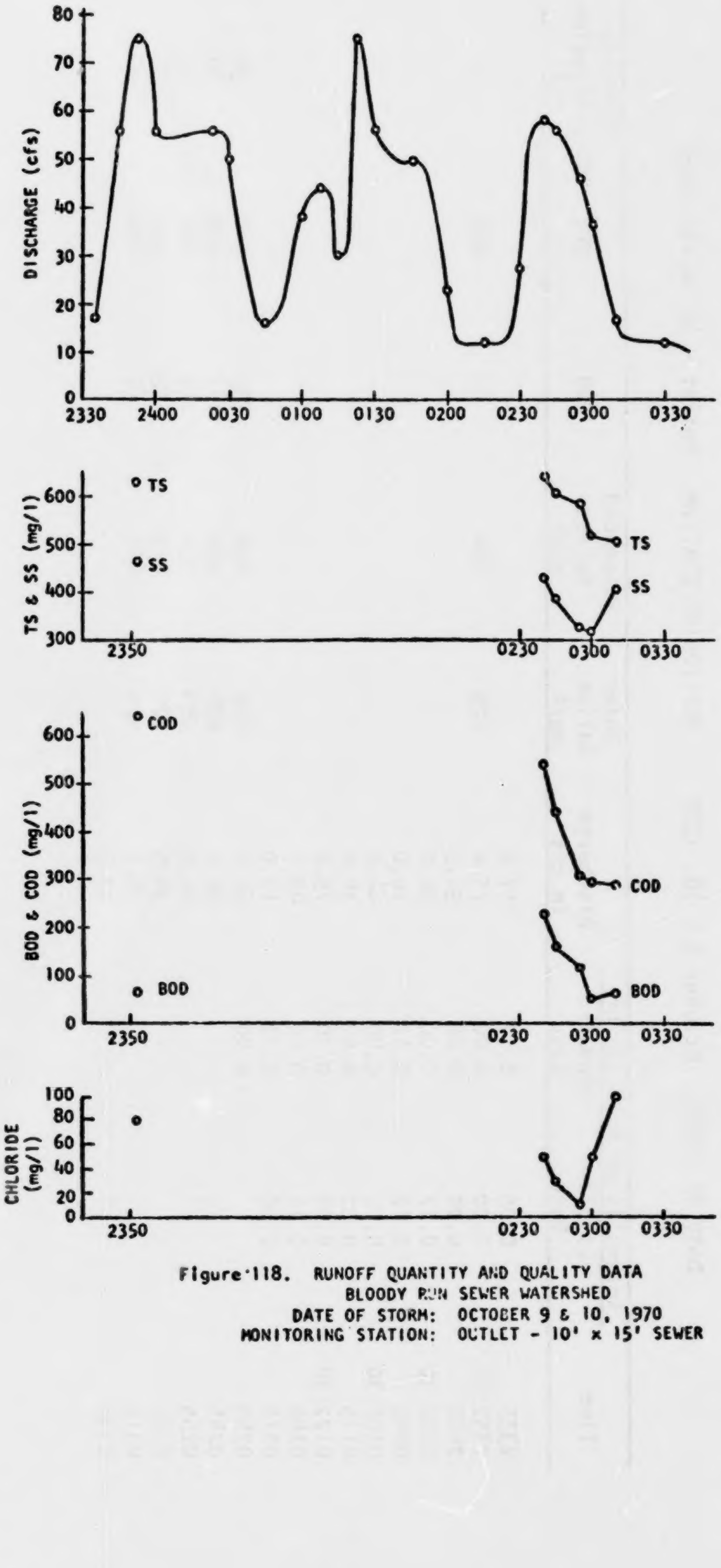


Figure 118. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: OCTOBER 9 & 10, 1970
MONITORING STATION: OUTLET - 10' x 15' SEWER

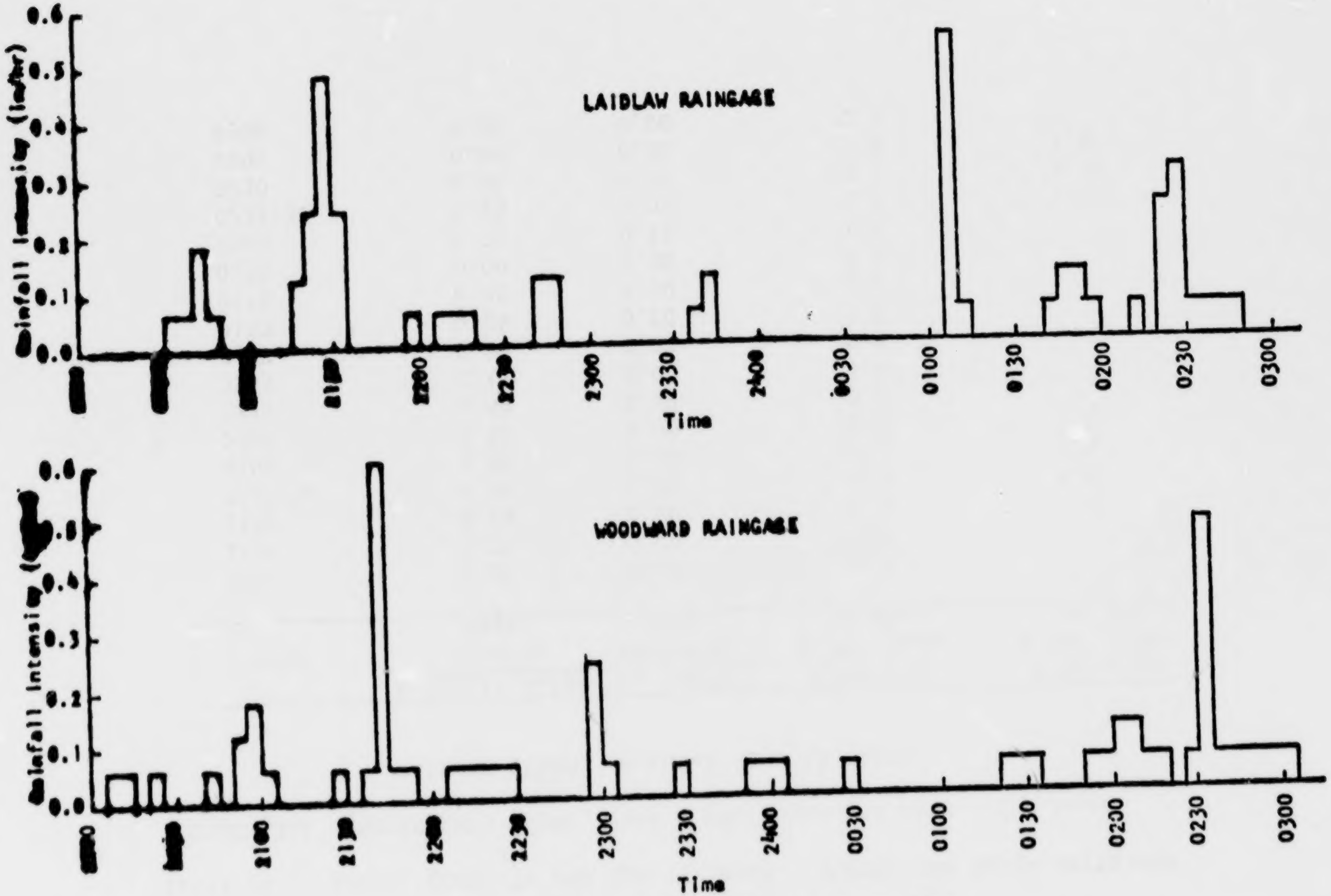


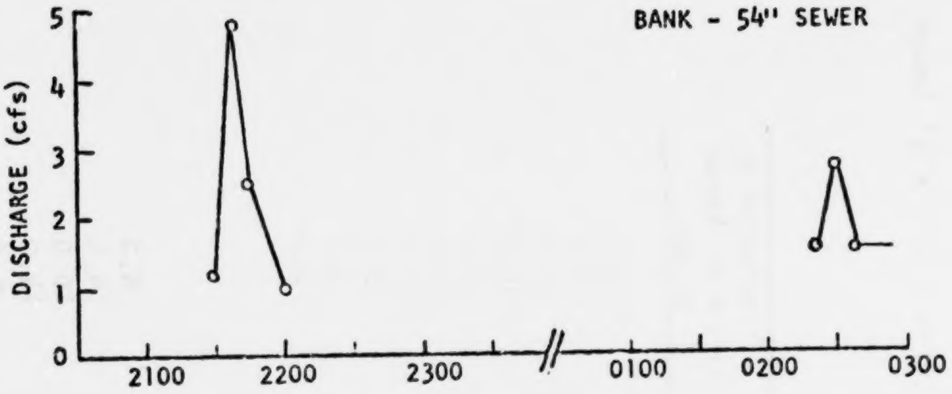
Figure 119. RAIN OF OCTOBER 12 & 13, 1970
HYETOGRAPHS

Table 54 . RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DISCHARGE MEASUREMENTS FROM: Bank - 78" Sewer and Bank - 54" Sewer

DATE OF STORM: October 12 & 13, 1970

Time	Rainfall Intensity (in/hr) Laidlaw Gage	Rainfall Intensity (in/hr) Woodward Gage	Discharge @ 78" Sewer in cfs	Discharge @ 54" Sewer in cfs
2045	0.06	0.00	2.2	
2100	0.00	0.06	1.9	
2130	0.24	0.00	10.2	1.2
2137:30	0.00	0.06	2.2	4.8
2145	0.00	0.60	2.5	2.5
2200	0.00	0.00	2.2	1.0
2230	0.00	0.00	0.6	
2245	0.12	0.00	1.6	
2300	0.00	0.24	0.9	
0107	0.54	0.00	1.6	
0115	0.00	0.00	2.2	
0130	0.00	0.06	0.5	
0200	0.00	0.12	1.6	
0222:30	0.24	0.00	5.0	1.5
0230	0.06	0.06	3.7	2.75
0245	0.06	0.06	1.6	1.5
0300	0.00	0.06	0.5	



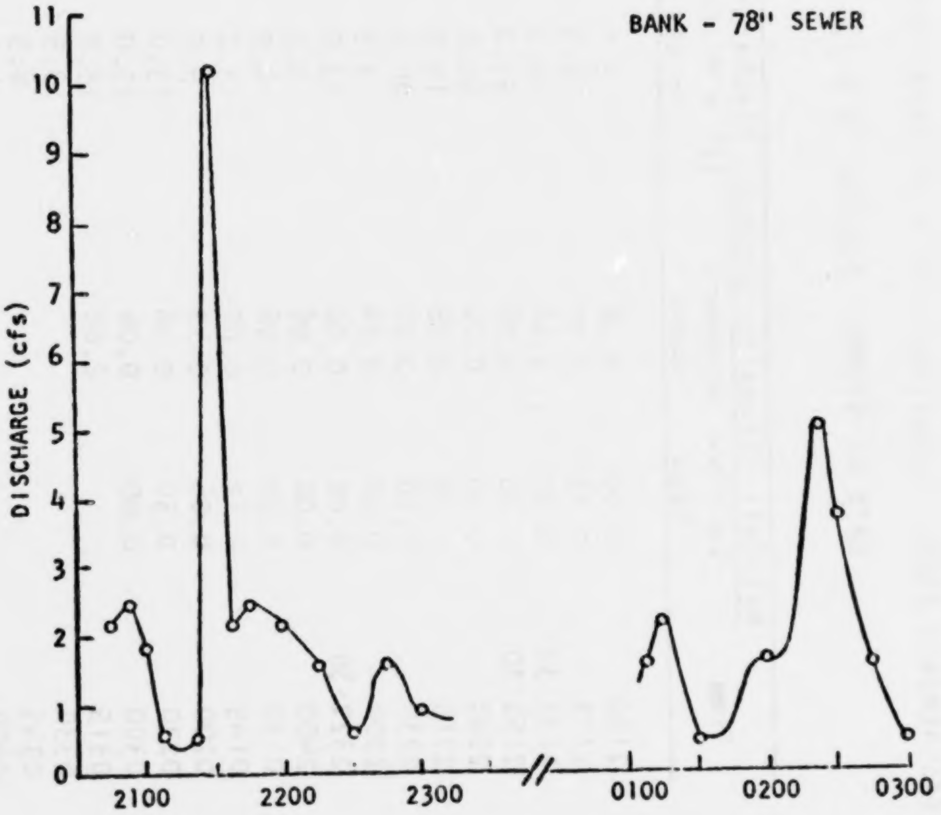


Figure 120. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: OCTOBER 12 & 13, 1970
DISCHARGE FROM BANK - 54" & 78" SEWER

Table 55 . RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DISCHARGE MEASUREMENTS FROM: Longview - 12' x 9' Sewer and Longview - 11' x 9' Sewer

DATE OF STORM: October 12 & 13, 1970

Time	Rainfall Intensity (in/hr) Laidlaw Gage	Rainfall Intensity (in/hr) Woodward Gage	Discharge @ 12' x 9' Sewer in cfs	Discharge @ 11' x 9' Sewer in cfs
2100	0.00	0.06	5.5	10.0
2115	0.12	0.00	16.5	6.0
2137:30	0.00	0.06	10.7	20.0
2152:30	0.00	0.06	55.0	20.0
2200	0.00	0.00	50.0	14.0
2215	0.06	0.06	19.5	35.0
2230	0.00	0.00	20.0	22.0
2300	0.00	0.24	9.5	18.5
2322:30	0.00	0.00	13.0	9.0
2400	0.00	0.06	5.2	10.3
0130	0.00	0.06	4.0	
0145	0.12	0.00	13.2	
0200	0.06	0.12	9.5	
0230	0.30	0.06	13.0	
0300	0.00	0.06	23.0	8.0
0315		0.00	15.0	14.0
0330			9.5	10.0
0345			6.3	14.0
0400			4.0	9.0

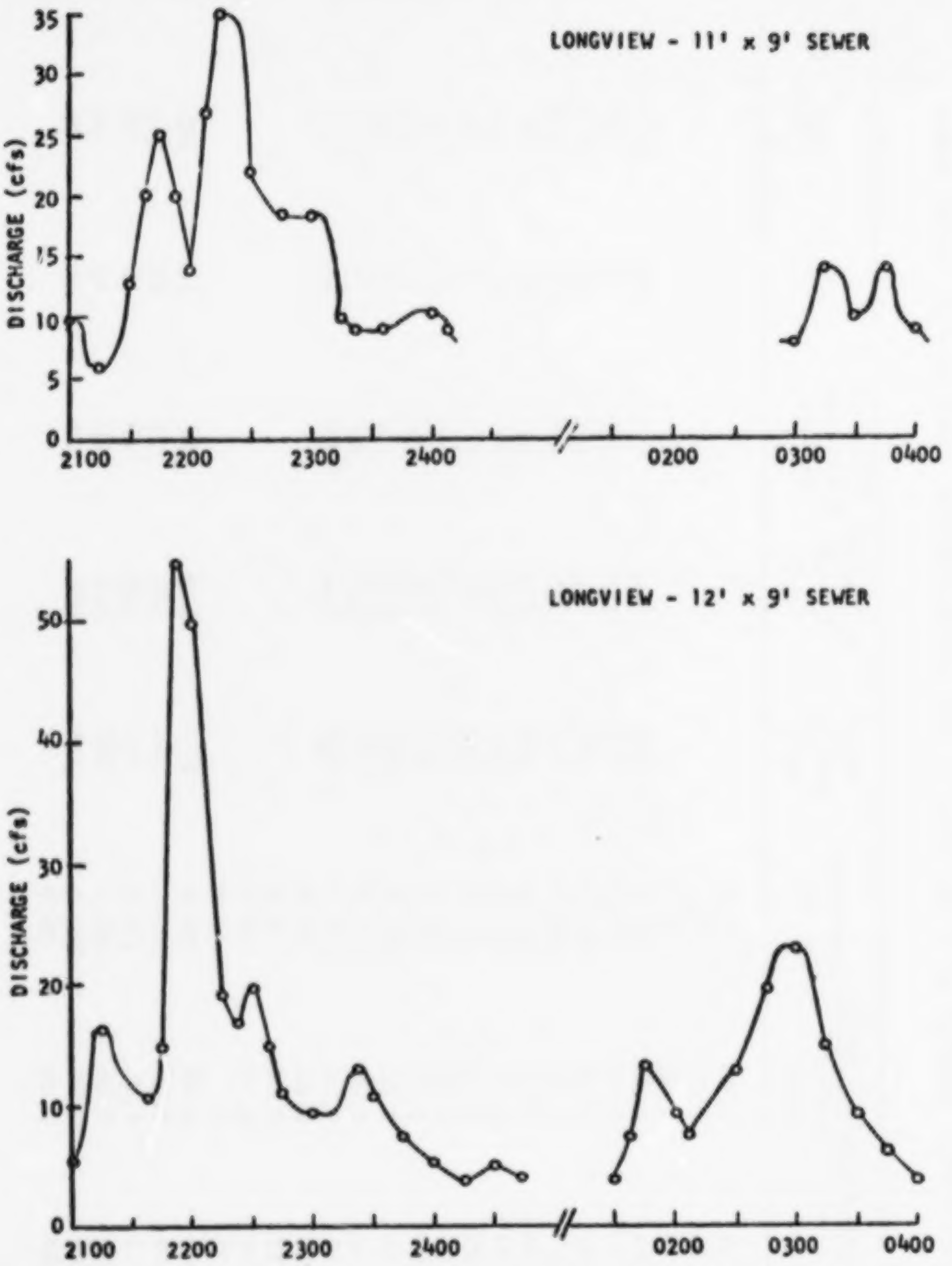


Figure 121. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: OCTOBER 12 & 13, 1970
DISCHARGE FROM LONGVIEW - 11' x 9' & 12' x 9' SEWER

Table 56 . RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: October 12 & 13, 1970 MONITORING STATION: Outlet - 10' x 15' Sewer

Time	Intensity of Rain (in/hr)		Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	Volatile Suspended Solids mg/l	BOD	COD	Chloride Concentration mg/l
	Laidlaw Gage	Woodward Gage							
2030	0.06	0.00	17.0						
2045	0.06	0.00	60.0						
2100	0.00	0.06	27.0	698	544	250	122	400	15
2122:30	0.24	0.00	370.0	686	561	242	116	392	33
2130	0.24	0.00	312.0	642	482	268	116	376	33
2137:30	0.00	0.06	300.0	622	424	272	120	388	19
2145	0.00	0.06	273.0	606	372	288	83	408	21
2152:30	0.00	0.06	504.0	584	324	244	63	192	25
2200	0.00	0.00	642.0	562	382	252	70	152	30
2207:30	0.06	0.06	347.0	548	349	238	83	135	30
2215	0.06	0.06	266.0	826	486	226	43	196	20
2222:30	0.00	0.06	182.0	884	529	212	70	196	15
2230	0.00	0.00	263.0	842	608	201	73	192	20
2237:30	0.00	0.00	275.0	898	624	189	63	189	30
2245	0.12	0.00	250.0						
2300	0.00	0.06	60.0						
0145	0.12	0.00	14.0						
0200	0.00	0.12	60.0						
0215	0.06	0.06	35.0	877	612	186	60	296	20
0237:30	0.06	0.48	120.0	748	586	172	83	200	10
0245	0.06	0.06	203.0	724	574	168	70	96	10
0252:30	0.00	0.06	370.0	707	528	168	40	96	10
0300	0.00	0.06	250.0	738	586	160	109	96	10
0307:30	0.00	0.00	247.0						
0330	0.00	0.00	76.0						

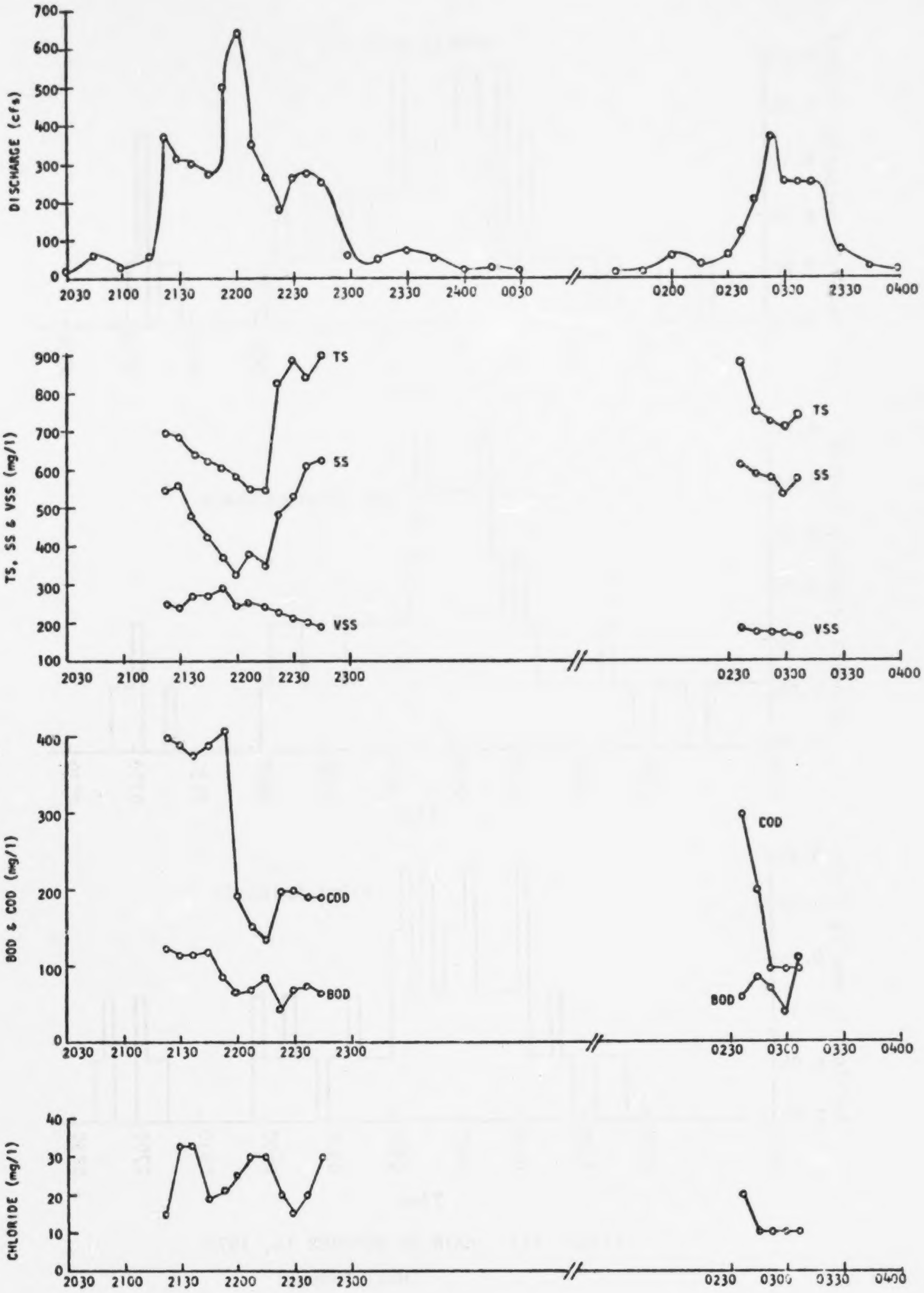


Figure 122. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: OCTOBER 12 & 13, 1970
MONITORING STATION: OUTLET - 10' x 15' SEWER

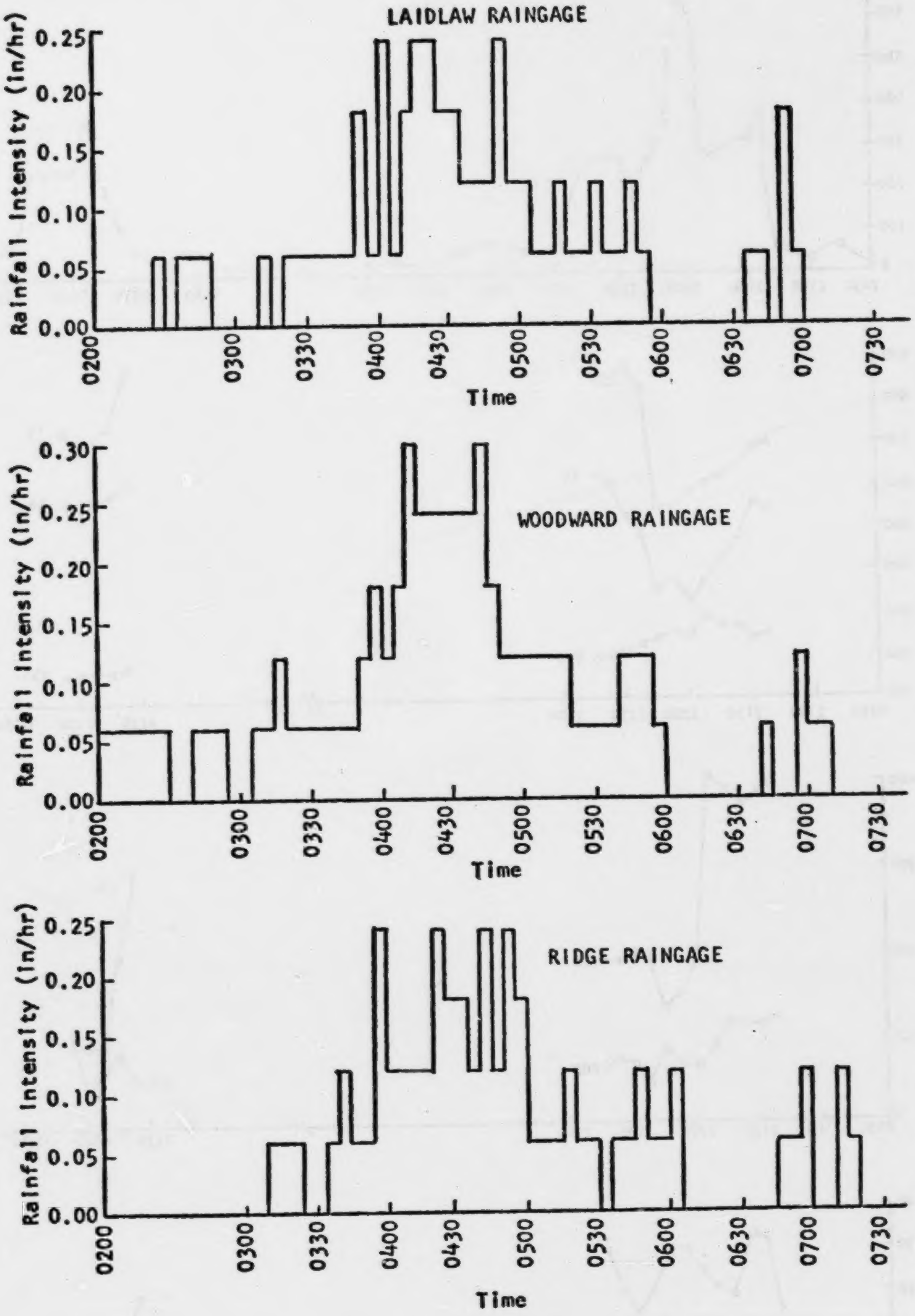


Figure 123. RAIN OF OCTOBER 14, 1970
HYETOGRAPHS

Table 57 . RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DISCHARGE MEASUREMENTS FROM: Bank - 78" Sewer and Bank - 54" Sewer

DATE OF STORM: October 14, 1970

Time	Rainfall Intensity (in/hr) Laidlaw Gage	Rainfall Intensity (in/hr) Woodward Gage	Rainfall Intensity (in/hr) Ridge Gage	Discharge @ 78" Sewer in cfs	Discharge @ 54" Sewer in cfs
0300	0.00	0.00	0.00	1.0	
0315	0.00	0.12	0.06	2.2	
0330	0.06	0.06	0.00	7.7	
0345	0.06	0.06	0.06	10.1	1.5
0400	0.24	0.12	0.12	6.0	3.1
0415	0.24	0.24	0.12	5.0	3.4
0430	0.18	0.24	0.18	3.2	7.9
0445	0.12	0.18	0.12	2.2	4.4
0500	0.12	0.12	0.06	2.2	3.1
0515	0.12	0.12	0.12	1.8	2.2
0530	0.12	0.06	0.00	1.0	1.5
0600	0.00	0.00	0.12	0.5	
0615	0.00	0.00	0.00	0.9	
0630	0.00	0.00	0.00	1.1	
0645	0.00	0.00	0.06	0.5	

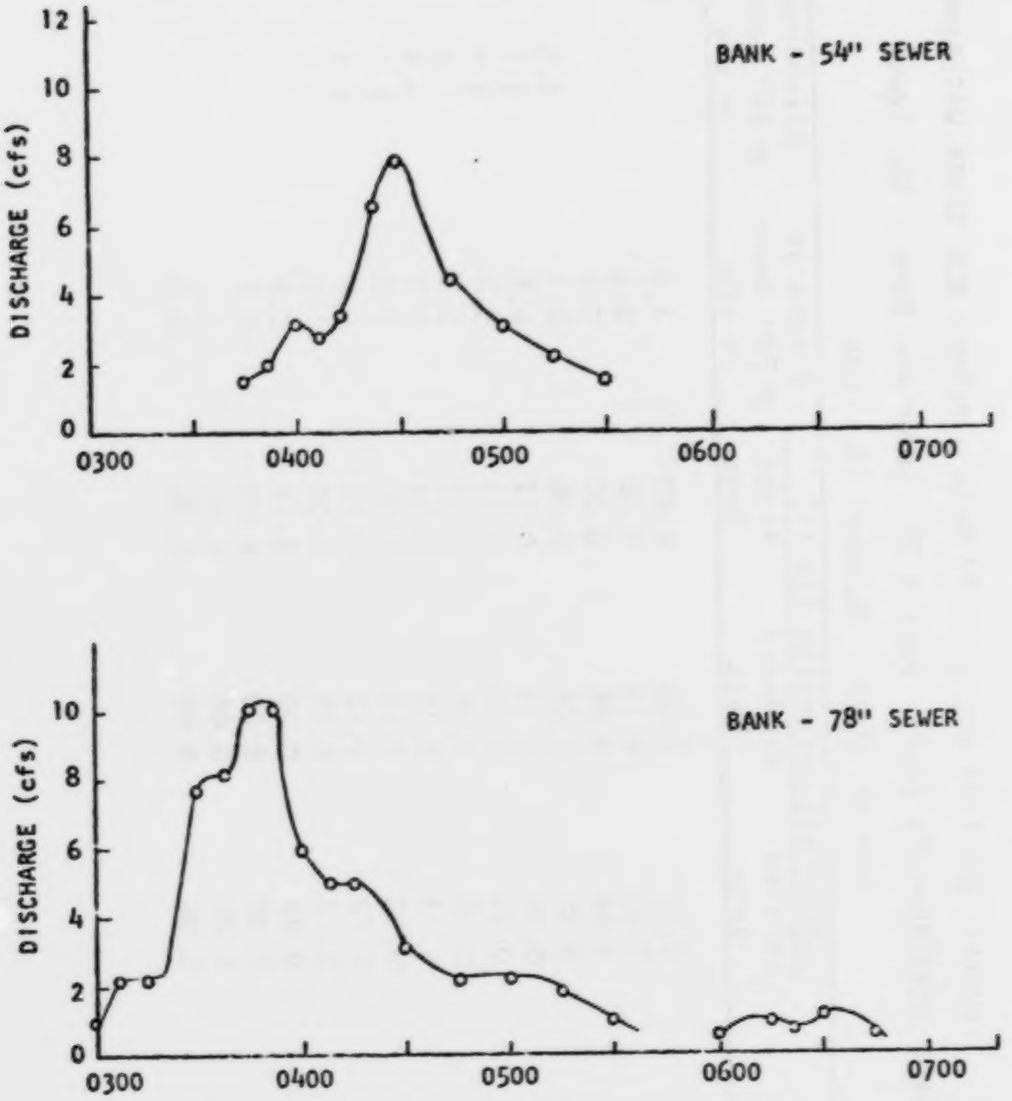


Figure 124. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: OCTOBER 14, 1970
DISCHARGE FROM BANK - 54" & 78" SEWER

Table 58. RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DISCHARGE MEASUREMENTS FROM: Longview - 12' x 9' Sewer and Longview - 11' x 9' Sewer

DATE OF STORM: October 14, 1970

Time	Rainfall Intensity (in/hr)			Discharge at 12' x 9' Sewer in cfs	Discharge at 11' x 9' Sewer in cfs
	Laidlaw Gage	Woodward Gage	Ridge Gage		
0400	0.24	0.12	0.12	7.0	
0415	0.24	0.24	0.12	19.5	
0430	0.18	0.24	0.18	31.5	14.0
0445	0.12	0.18	0.12	44.0	20.0
0500	0.12	0.12	0.06	44.0	27.0
0515	0.12	0.12	0.12	35.7	33.8
0530	0.12	0.06	0.00	29.0	25.0
0545	0.12	0.12	0.12	21.5	18.2
0600	0.00	0.00	0.12	19.5	15.3
0615	0.00	0.00	0.00	16.5	14.0
0630	0.00	0.00	0.00	14.7	15.3
0645	0.00	0.00	0.06	7.0	12.5
0700	0.00	0.06	0.00	6.5	
0715		0.00	0.06	10.7	
0730			0.00	13.3	8.8
0800				7.0	8.5
0830				5.3	

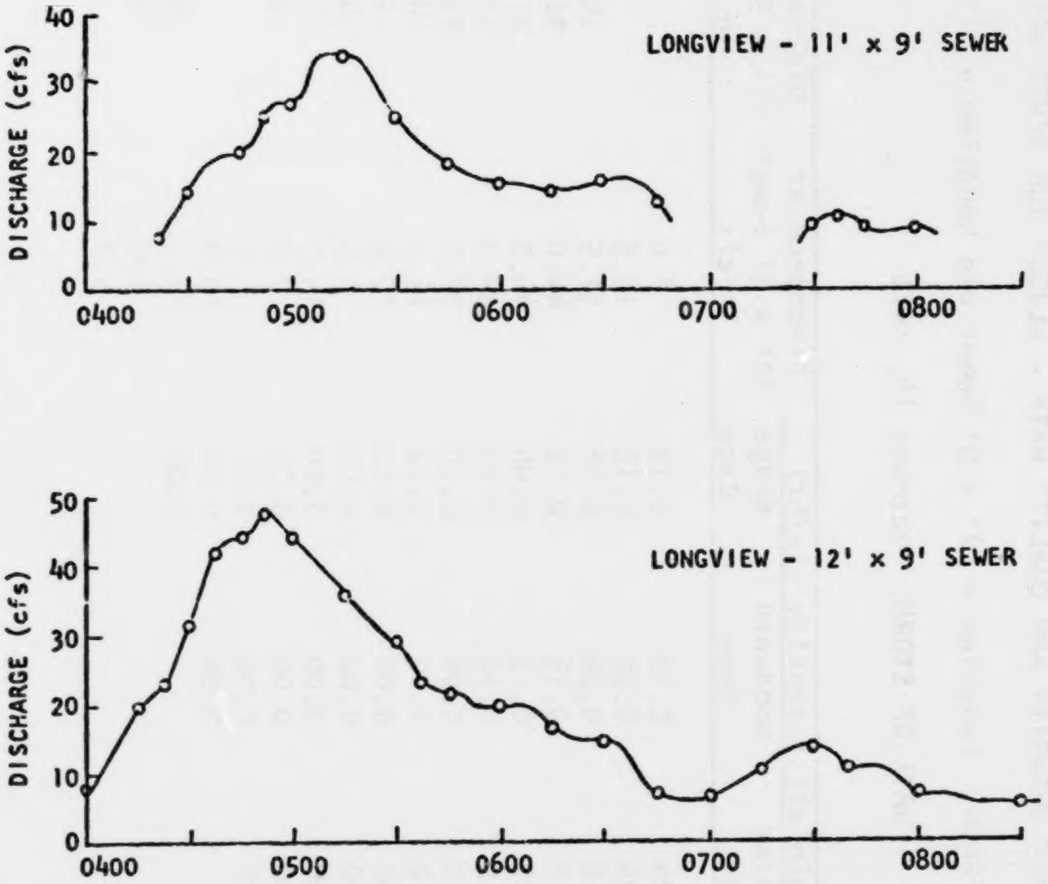


Figure 125. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: OCTOBER 14, 1970
DISCHARGE FROM LONGVIEW - 11' x 9' & 12' x 9' SEWERS

Table 59 . RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: October 14, 1970 MONITORING STATION: Outlet - 10' x 15' Sewer

Time	Rainfall Intensity (in/hr)			Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	BOD	COD	Chloride Concentration mg/l
	Laidlaw Gage	Woodward Gage	Ridge Gage						
0355	0.06	0.18	0.24	100.0	848	428	122	200	15
0402:30	0.24	0.12	0.12	370.0	872	444	128	152	10
0410	0.18	0.30	0.12	372.0	888	462	114	140	20
0417:30	0.24	0.24	0.12	374.0	892	480	114	134	8
0425	0.18	0.24	0.18	382.0	964	528	128	116	10
0432:30	0.18	0.24	0.18	560.0	998	564	110	124	11
0440	0.12	0.30	0.24	320.0	984	540	70	132	9
0447:30	0.12	0.18	0.12	250.0	920	521	65	112	12
0455	0.24	0.12	0.18	252.0	872	488	50	100	10
0502:30	0.12	0.12	0.06	250 0	806	472	50	100	8
0510	0.06	0.12	0.06	250.0	748	438	50	100	10
0517:30	0.12	0.12	0.12	243.0	707	386	65	112	9
0525	0.06	0.06	0.06	150.0	684	372	75	140	8
0535	0.06	0.06	0.06	200.0	632	348	81	128	10
0542:30	0.06	0.12	0.06	254.0	648	389	90	132	10
0550	0.06	0.12	0.06	252.0	602	322	65	140	10
0557:30	0.00	0.06	0.06	250.0	542	306	50	144	10
0605	0.00	0.00	0.00	250.0	516	300	45	136	10
0630	0.00	0.00	0.00	40.0					
0700	0.00	0.06	0.00	55.0					
0730	0.00	0.00	0.00	42.0					
0800	0.00	0.00	0.00	18.0					

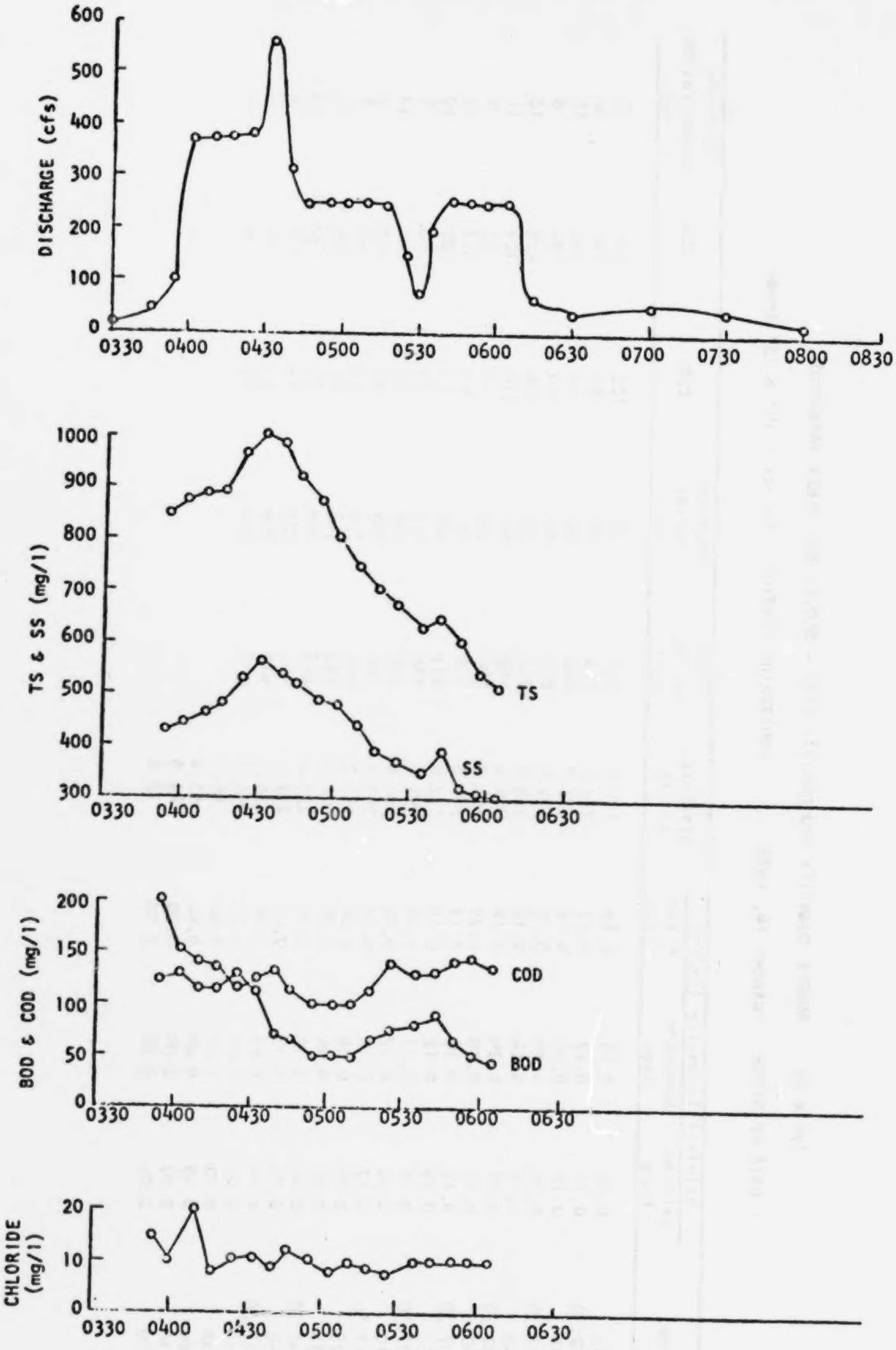


Figure 126. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: OCTOBER 14, 1970
MONITORING STATION: OUTLET - 10' x 15' SEWER

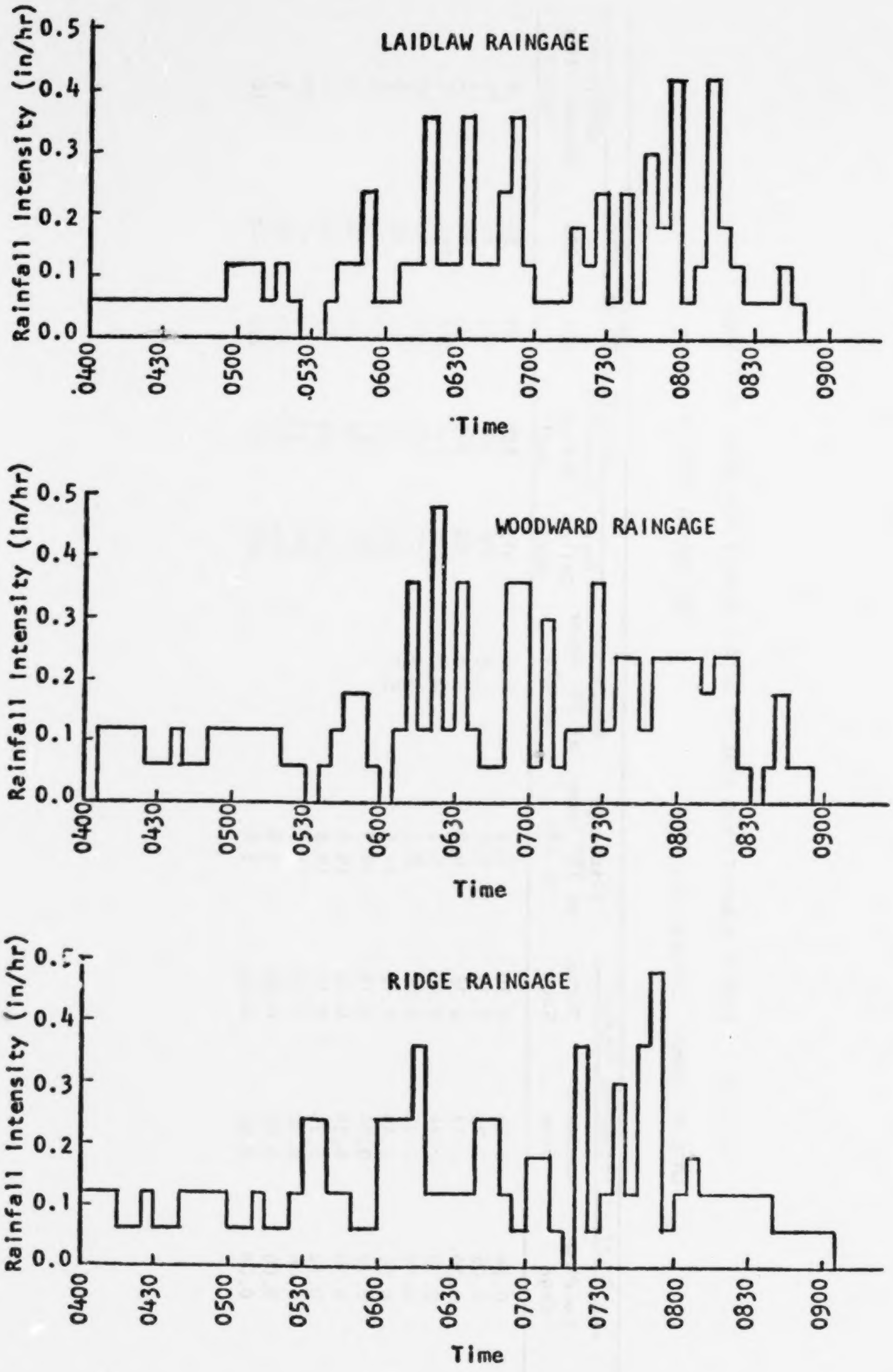


Figure 127. RAIN OF OCTOBER 20, 1970
HYETOGRAPHS

Table 60 . RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: October 20, 1970 MONITORING STATION: Bank

Time	Rainfall Intensity (in/hr)			Discharge at 78" Sewer in cfs	Discharge at 54" Sewer in cfs	Samples from 54" Sewer				
	Laidlaw Gage	Woodward Gage	Ridge Gage			Total Solids mg/l	Suspended Solids mg/l	BOD	COD	Chloride Concentration mg/l
0735	0.24	0.24	0.30	7.5	3.4	412	228	67	84	6
0740	0.06	0.24	0.12	5.0	3.5	406	264	103	88	11
0745	0.30	0.12	0.36	4.0	3.8	360	245	93	56	10
0750	0.18	0.24	0.48	5.7	4.1	342	216	216	104	8
0755	0.42	0.24	0.06	8.5	4.7	320	188	150	72	10
0800	0.06	0.24	0.12	11.5	5.7	308	176	107	68	9
0807:30	0.12	0.24	0.18	18.5		292	165	57	40	9
0815	0.18	0.24	0.12	14.4		264	146	57	44	8
0822:30	0.12	0.24	0.12	10.0		248	134	57	40	10
0830	0.06	0.00	0.12	3.7		226	128	60	40	10
0845	0.06	0.06	0.06	5.5		214	124	213	56	6
0900	0.00	0.00	0.06	3.6		208	120	226	42	10

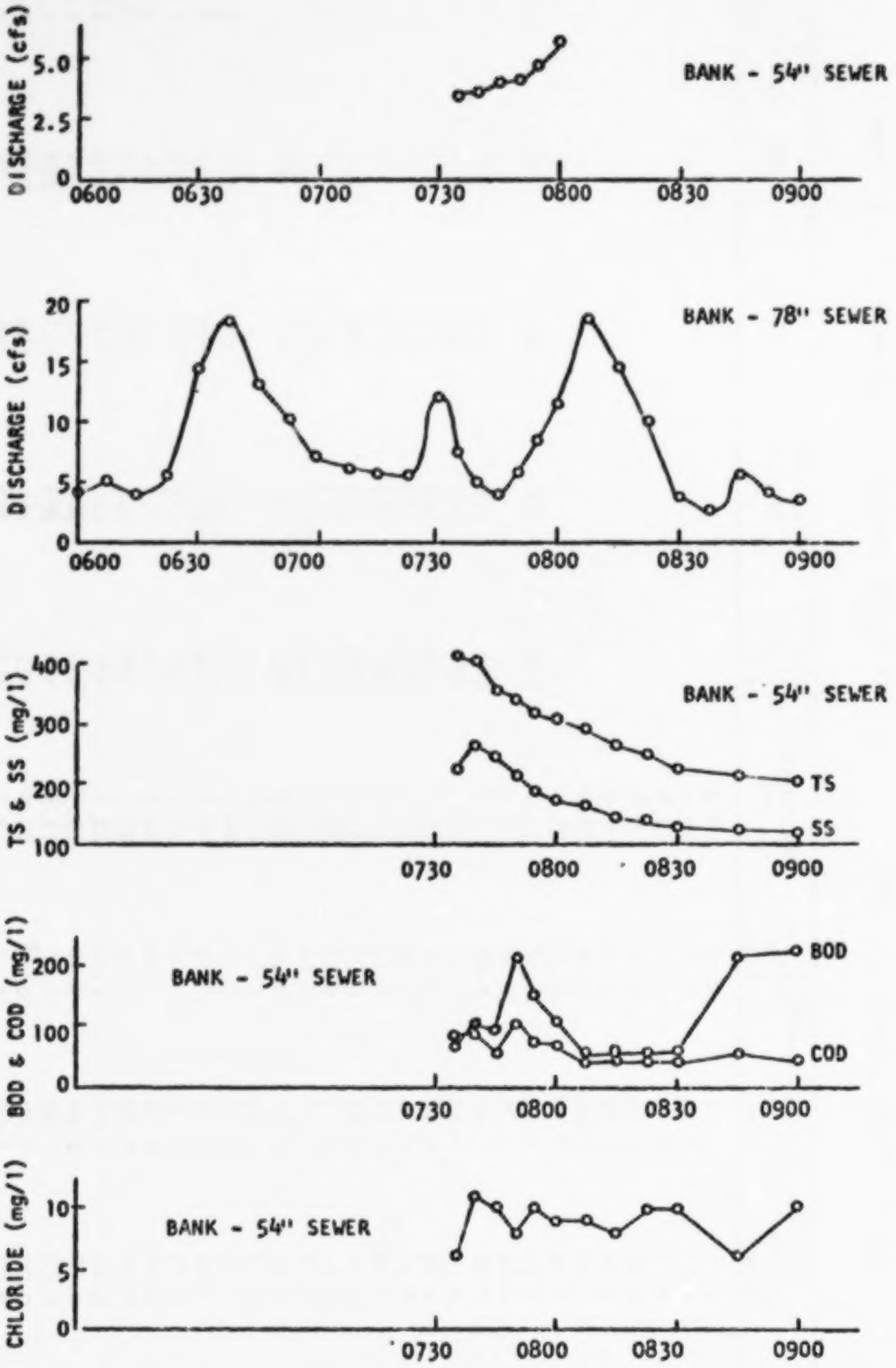


Figure 128. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: OCTOBER 20, 1970
MONITORING STATION: BANK - 54" & 78" SEWER

Table 61 RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: October 20, 1970 MONITORING STATION: Longview - 11' x 9' Sewer

Time	Rainfall Intensity (in/hr)			Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	BOD	COD	Chloride Concentration mg/l
	Laidlaw Gage	Woodward Gage	Ridge Gage						
0500	0.12	0.12	0.06	9.0					
0530	0.00	0.00	0.24	12.6					
0545	0.12	0.18	0.12	10.3					
0600	0.06	0.00	0.24	18.0					
0630	0.36	0.36	0.12	39.0					
0645	0.24	0.06	0.24	39.0					
0652:30	0.36	0.36	0.12	39.0					
0700	0.06	0.06	0.18	32.6	628	348	246	792	0.1
0705	0.06	0,30	0.18	38.2					
0710	0.06	0.06	0.06	37.0	640	386	190	968	1
0715	0.18	0.12	0.00	27.0	684	394	196	328	1
0720	0.12	0.12	0.36	25.5	725	426	185	252	1
0725	0.24	0.36	0.06	25.3	700	420	196	248	1
0730	0.06	0.12	0.12	27.0	786	468	203	204	1
0735	0.24	0.24	0.30	31.5	744	524	183	200	1.2
0745	0.30	0.12	0.36	26.1	680	485	173	148	1
0750	0.18	0.24	0.48	30.5	642	450	163	68	1
0755	0.42	0.24	0.06	35.0	602	432	152	160	1
0805	0.12	0.24	0.18	41.0					
0825	0.06	0.06	0.12	44.0	585	426	185	252	1
0830	0.06	0.00	0.12	41.5	548	384	203	212	1
0840	0.12	0.18	0.06	35.5	526	372	176	104	1
0855	0.00	0.00	0.06	25.0	512	350	146	96	1.3
0900	0.00	0.00	0.06	21.5	488	326	156	80	1.4
0910	0.00	0.00	0.00	22.5	476	316	152	216	1.3
0920	0.00	0.00	0.00	19.5	460	304	150	240	1.5
0930	0.00	0.00	0.00	12.5	444	285	150	224	1.6
0940	0.00	0.00	0.00	10.0	422	264	145	248	1.4
0950	0.00	0.00	0.00	8.7	408	252	150	284	2.3
1005	0.00	0.00	0.00	7.0	378	248	156	332	2.7

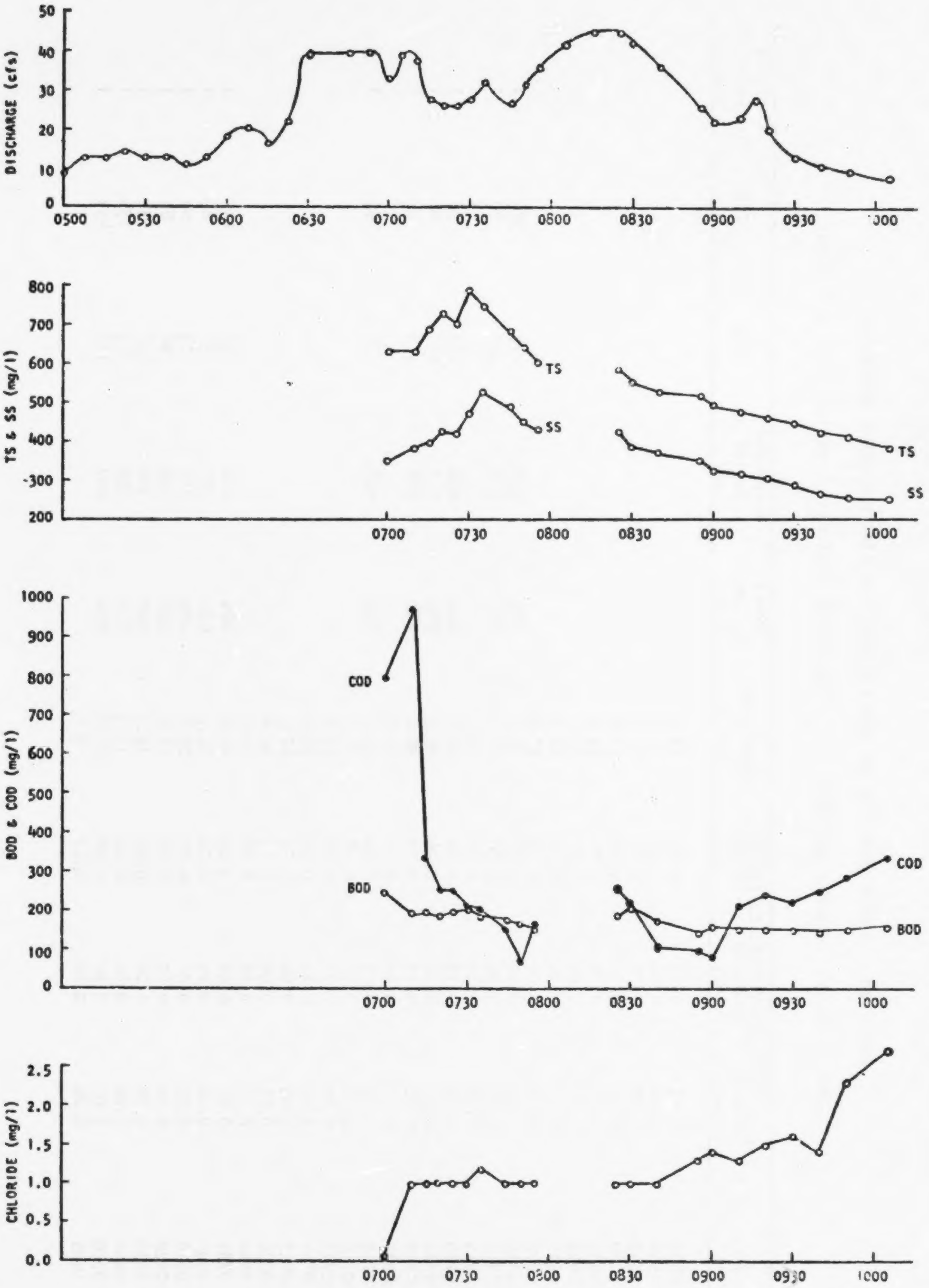


Figure 129. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: OCTOBER 20, 1970
MONITORING STATION: LONGVIEW - 11' x 9' SEWER

Table 62. RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: October 20, 1970 MONITORING STATION: Longview - 12' x 9' Sewer

Time	Rainfall Intensity (in/hr)			Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	BOD	COD	Chloride Concentration mg/l
	Laidlaw Gage	Woodward Gage	Ridge Gage						
0500	0.12	0.12	0.06	13.0					
0530	0.00	0.00	0.24	19.5					
0545	0.12	0.18	0.12	15.0					
0600	0.06	0.00	0.24	27.2					
0630	0.36	0.36	0.12	42.0					
0645	0.24	0.06	0.24	55.0					
0625:30	0.36	0.36	0.12	63.0					
0700	0.06	0.06	0.18	55.0					
0705	0.06	0.30	0.18	50.5	684	468	173	284	1
0710	0.06	0.06	0.06	50.5	735	492	143	80	1
0715	0.18	0.12	0.00	52.5					
0720	0.12	0.12	0.36	46.0	844	546	123	96	1
0725	0.24	0.36	0.06	40.0	816	502	110	88	1
0730	0.06	0.12	0.12	38.0	758	492	104	56	1
0735	0.24	0.24	0.30	42.5					
0745	0.30	0.12	0.36	50.0	712	428	87	92	1
0750	0.18	0.24	0.48	50.0					
0755	0.42	0.24	0.06	52.0					
0805	0.12	0.24	0.18	55.0					
0825	0.06	0.06	0.12	62.7					
0830	0.06	0.00	0.12	60.6					
0840	0.12	0.18	0.06	54.0					
0855	0.00	0.00	0.06	40.0	664	404	77	152	1
0900	0.00	0.00	0.06	32.2	630	380	57	68	1
0910	0.00	0.00	0.00	27.5	605	342	73	64	1
0920	0.00	0.00	0.00	21.8	588	368	90	80	1
0930	0.00	0.00	0.00	16.4	550	300	107	132	1
0940	0.00	0.00	0.00	14.3	524	288	123	196	1
0950	0.00	0.00	0.00	11.5	484	242	110	452	1
1005	0.00	0.00	0.00	9.4					

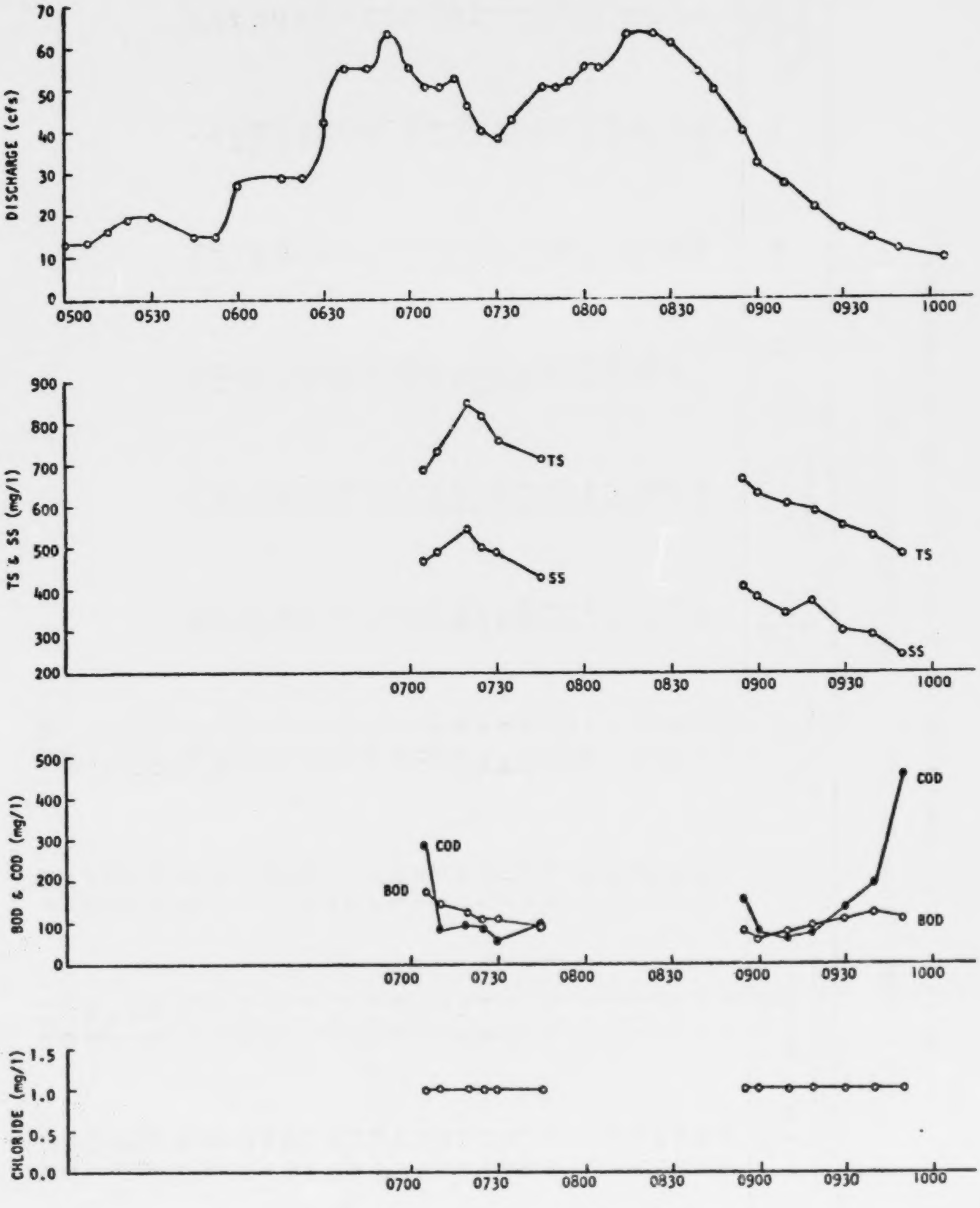


Figure 130. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: OCTOBER 20, 1970
MONITORING STATION: LONGVIEW - 12' x 9' SEWER

Table 63 . RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: October 20, 1970 MONITORING STATION: Outlet - 10' x 15' Sewer

Time	Rainfall Intensity (in/hr)			Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	Volatile Suspended Solids mg/l	BOD	COD	Chloride Concentration mg/l
	Laidlaw Gage	Woodward Gage	Ridge Gage							
0515	0.12	0.12	0.06	48.0						
0522:30	0.06	0.06	0.06	154.0	624	484	226	266	1192	30
0530	0.00	0.00	0.24	154.0	672	492	274	266	1184	8
0537:30	0.06	0.06	0.24	155.0	684	528	256	250	1016	10
0545	0.12	0.12	0.12	153.0	695	564	264	186	568	22
0552:30	0.24	0.18	0.06	115.0	742	582	288	203	744	13
0600	0.06	0.00	0.24	252.0	788	604	292	176	680	20
0607:30	0.12	0.12	0.24	252.0	865	608	298	126	200	9
0615	0.36	0.12	0.36	251.0	898	626	304	140	184	10
0622:30	0.12	0.48	0.12	202.0	954	642	300	136	232	12
0630	0.36	0.36	0.12	201.0	922	610	295	130	216	9
0637:30	0.12	0.12	0.12	201.0	910	576	286	126	144	10
0645	0.24	0.06	0.24	154.0	888	555	254	133	136	8
0652:30	0.36	0.36	0.12	154.0	845	540	228	110	136	9
0700	0.06	0.06	0.18	154.0	824	532	216	116	128	12
0707:30	0.06	0.30	0.18	154.0	786	507	202	116	80	11
0715	0.18	0.12	0.00	154.0	770	470	198	67	80	10
0722:30	0.12	0.12	0.36	154.0	752	442	192	87	120	1
0730	0.06	0.12	0.12	154.0	648	388	185	90	136	2
0737:30	0.24	0.24	0.30	153.0	662	360	180	50	256	22
0745	0.06	0.12	0.36	153.0	628	345	172	90	248	32
0752:30	0.18	0.24	0.48	153.0	604	332	164	103	248	41
0800	0.06	0.24	0.12	153.0	589	324	160	110	256	42
0807:30	0.12	0.24	0.18	202.0	572	320	156	110	196	50
0815	0.18	0.24	0.12	202.0	584	335	152	107	-	32
0830	0.06	0.00	0.12	203.0						
0900	0.00	0.00	0.06	154.0						
0915	0.00	0.00	0.00	115.0						
0930	0.00	0.00	0.00	154.0						
0945	0.00	0.00	0.00	70.0						
1000	0.00	0.00	0.00	54.0						
1030	0.00	0.00	0.00	30.0						

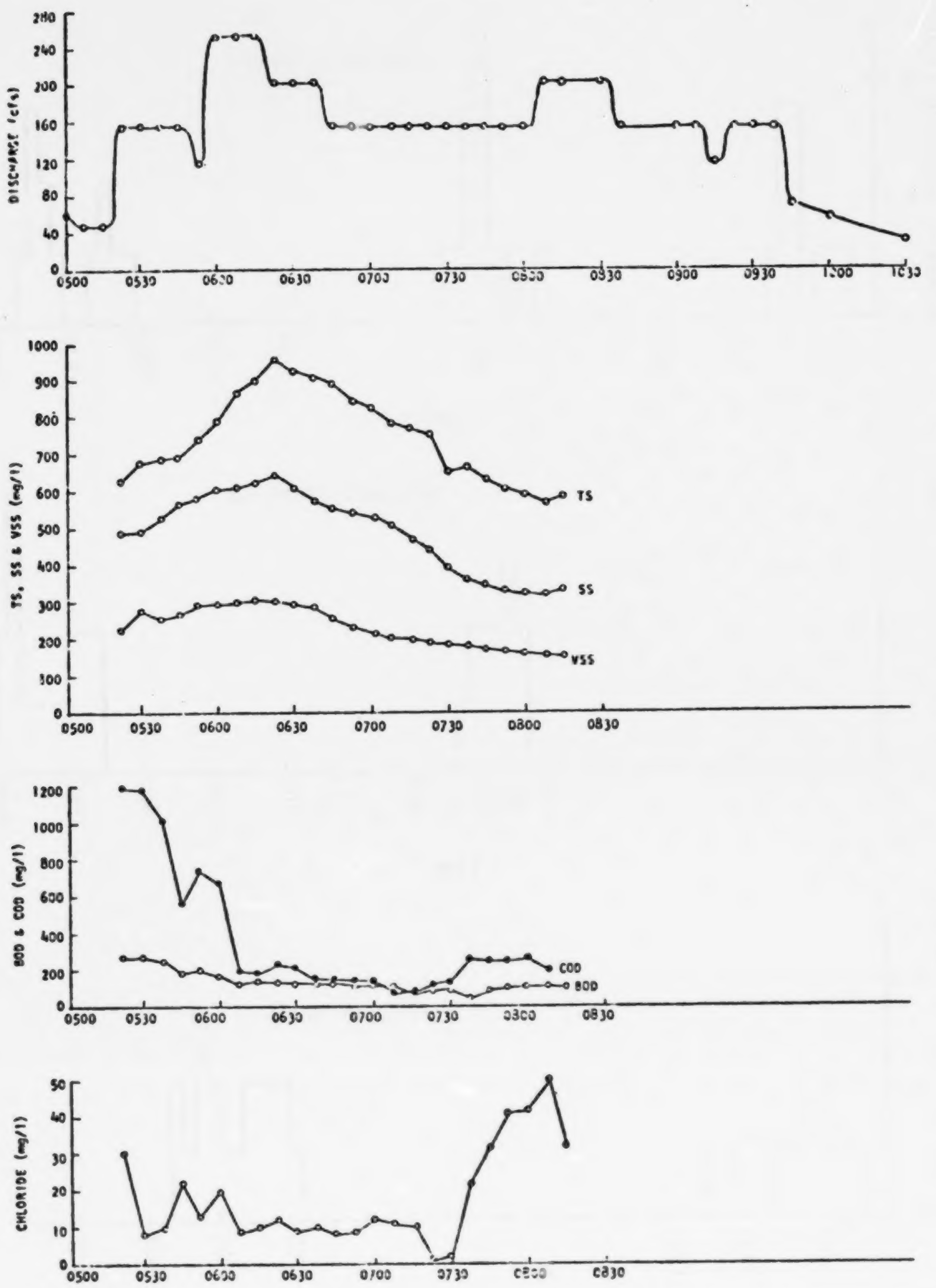


Figure 131. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM OCTOBER 20, 1970
MONITORING STATION: OUTLET - 10' x 15' SEWER

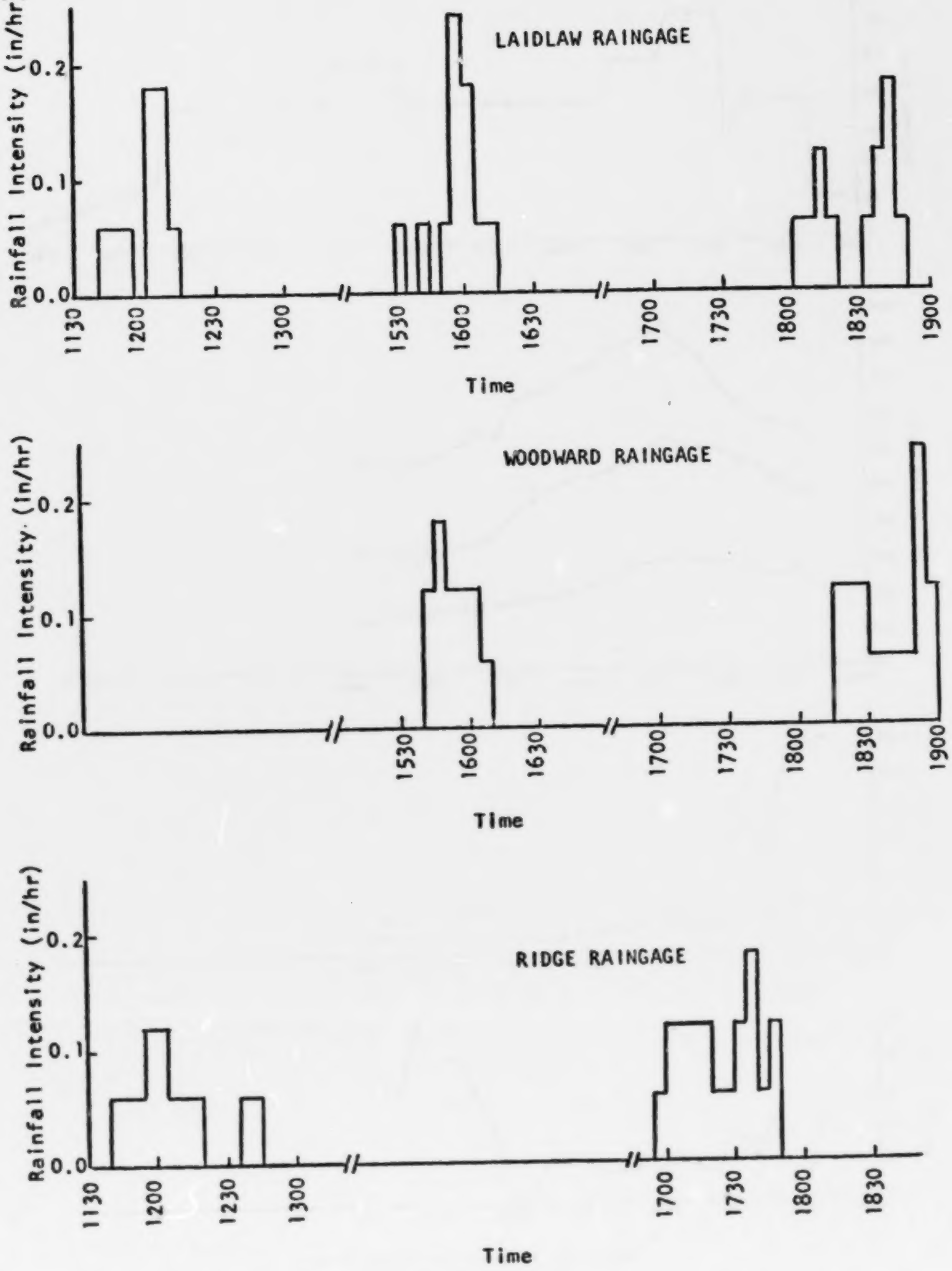


Figure 132. RAIN OF OCTOBER 20 (AFTERNOON), 1970
HYETOGRAPHS

Table 64 - RUNOFF QUANTITY AND QUALITY DATA - BLOODY RUN SEWER WATERSHED

DATE OF STORM: October 20 (afternoon), 1970 MONITORING STATION: Outlet - 10' x 15' Sewer

Time	Rainfall Intensity (in/hr)			Discharge in cfs	Total Solids mg/l	Suspended Solids mg/l	BOD	COD	Chloride Concen-tration
	Laidlaw Gage	Woodward Gage	Ridge Gage						
1600	0.18	0.12	0.00	26.0					
1615	0.06	0.00	0.00	200.0	542	388	186	232	163
1622:30	0.00	0.00	0.00	154.0	628	392	211	256	153
1630	0.00	0.00	0.00	153.0	706	464	220	272	153
1637:30	0.00	0.00	0.00	113.0					
1645	0.00	0.00	0.00	48.0					
1700	0.00	0.00	0.12	35.0					
1730	0.00	0.00	0.12	27.0					
1800	0.06	0.00	0.00	48.0					
1830	0.06	0.12	0.00	61.0					
1837:30	0.12	0.06	0.00	202.0	684	324	232	264	163
1845	0.06	0.06	0.00	154.0	626	316	236	256	144
1852:30	0.00	0.24	0.00	154.0	588	302	216	228	134
1900	0.00	0.12	0.00	154.0	570	283	186	192	106
1907:30	0.00	0.00	0.00	153.0	542	264	143	192	106
1915	0.00	0.00	0.00	153.0	528	292	130	172	104
1922:30	0.00	0.00	0.00	153.0	489	248	130	180	96
1930	0.00	0.00	0.00	153.0	465	220	250	940	96
1945	0.00	0.00	0.00	54.0					
2000	0.00	0.00	0.00	48.0					
2030	0.00	0.00	0.00	35.0					
2100	0.00	0.00	0.00	14.0					

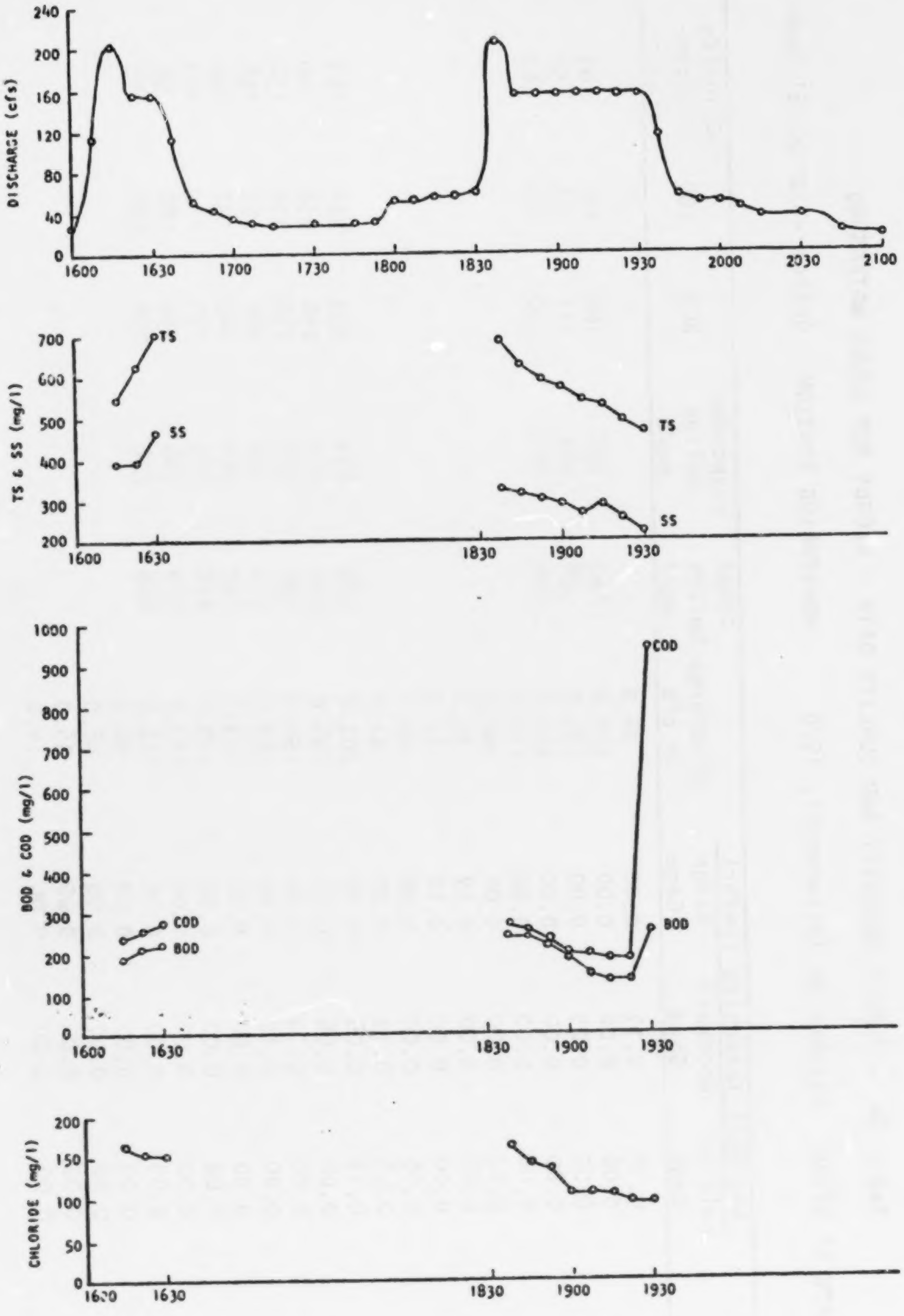


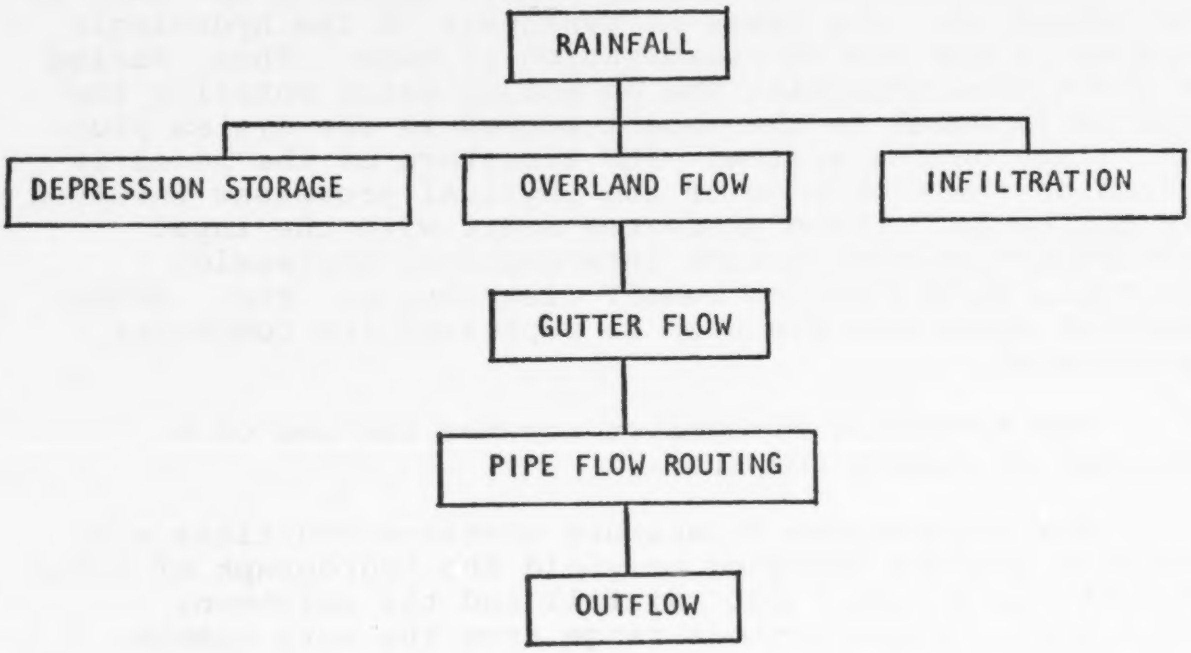
Figure 133. RUNOFF QUANTITY AND QUALITY DATA
BLOODY RUN SEWER WATERSHED
DATE OF STORM: OCTOBER 20 (AFTERNOON), 1970
MONITORING STATION: OUTLET - 10' x 15' SEWER

SECTION VI

MATHEMATICAL MODEL FOR SIMULATION OF HYDROLOGIC RUNOFF FROM AN URBAN WATERSHED

1. INTRODUCTION

The purpose of this mathematical model, termed herein as Cincinnati Urban Runoff Model, is to provide a quick and reliable method for obtaining the hydrograph of runoff at any selected point in a sewer system for any given rainfall. This is accomplished by simulating separately the processes involved during a rainfall: infiltration, depression storage, overland flow, gutter flow, and routing through the sewers. The Cincinnati Urban Runoff Model has been developed by using independent hydraulic and hydrologic principles and equations, and by combining the above processes into the sequential flow diagram below:



It is believed that the output of this mathematical model may be helpful in the management of existing sewer systems. It also should be useful in the design of new sewer systems.

2. REVIEW OF THE LITERATURE

System simulation techniques fall into two principal categories: deterministic methods and stochastic methods.

The deterministic methods attempt to develop relationships among the physical parameters and processes involved in the hydrologic cycle based on recorded data. These relationships are then used to generate or to predict non-recorded hydrologic sequences. The principal input function is rainfall, and the principal output function is runoff.

The stochastic methods use statistical measures of hydrologic variables to generate non-recorded sequences to which evaluation of probability levels are attached. Sufficient long-term records of the variables are required to give a true representation of their statistical nature.

Deterministic approaches can be subdivided into both parametric system synthesis and transformation methods.

The parametric system synthesis approach has much to recommend it. The basis of synthesis of the hydrologic system is the law of conservation of mass. Thus, during a given time interval, the amount of water entering the system is equal to the amount stored in the system plus that leaving the system. The structure of the model is based on the knowledge of the physical processes involved in the cycle. These processes start with the input precipitation and include interception, depression storage, infiltration, runoff distribution, etc. Mathematical equations are used to represent the component processes.

The foregoing discussion assumes the use of a digital or analog computer.

The engineering literature contains countless analytical methods designed to yield the hydrograph of urban runoff, given the input rainfall and the catchment structure. These methods range from the very simple Rational Method to the very detailed Chicago Hydrograph Method. To a varying degree, all methods rely upon empirical relationships and experience rather than upon the basic differential equations characterizing the flow. An example is the Rational Method based on the equation $Q = ciA$, known also as the Lloyd-Davies Method in the United Kingdom.

In 1932, Gregory and Arnold [1] developed a modification of the Rational Method to take in consideration catchment shape and slope.

In 1944, Hicks [2] developed a method of computing urban runoff for the Los Angeles area based on experimental work for the determination of the principal abstractions from rainfall and actual gaging of local drainage areas in the metropolitan region.

In 1946, Izzard [3] developed a method to simulate flow on a paved surface.

In 1960, extensive studies of a hydrograph method were made by the City of Chicago. A detailed explanation of this hydrograph type of analysis was presented by Tholin and Keifer [4]. This method simulates flow on grassed areas, roofs, paved streets, and through gutters and storm drains to the basin outlet. The method provides separate hydrographs representing the impervious areas and the pervious or grassed areas. The runoff hydrograph from the impervious area of an urban basin dominates the runoff regime and provides the peak flows which must be used in designing a storm drain system.

In 1963, Kaltenbach [5] summarized the Inlet Method of approach studied at the Johns Hopkins University. The method determines the flow to each inlet, attenuates the peak flow from each subarea as it moves down the pipe, and sums the attenuated peaks to determine the total peak at the design point.

For an existing urban basin, a unit hydrograph can be developed if observations are made of actual storm rainfalls and runoff. Eagleson [6] has discussed the unit hydrograph method, mentioning the utility of such unit hydrographs and the degree to which their characteristics can be related to properties of the urban basin.

The RRL (Road Research Laboratory) Method [7] is now widely used in Great Britain. The RRL Method uses only the impervious areas of a catchment connected to the storm drainage system. One important feature making the RRL Method superior to the inlet hydrograph method is that the RRL Method is readily applicable to basins before urban development takes place, so the storm drainage system can be adequately designed prior to construction.

The Stanford Watershed Model IV has been developed for the simulation of runoff from unsewered watersheds larger than urban basins [8]. Because a non-urban watershed is more pervious than an urban area, the Stanford Model properly gives greater emphasis to the moisture content of the soil and the movement of ground water with no consideration to routing through urban gutters and sewers.

Under the sponsorship of the Environmental Protection Agency, Water Quality Office, a consortium of contractors--Metcalf & Eddy, Inc., the University of Florida, and Water Resources Engineers, Inc., [9], have developed a comprehensive mathematical model, termed herein as EPA Storm Water Management Model, capable of representing urban storm water runoff and combined sewage overflow phenomena. Correctional devices in the form of user selected options for storage and/or treatment are provided with associated estimates of cost. Effectiveness is portrayed by computed treatment efficiencies and modeled changes in receiving water quality. This model resembles several of the previous methods, but it provides for more detailed handling of input parameters. The solution of a particular problem is accomplished on a high speed digital computer. Further testing of this model will be accomplished under the current project at the University of Cincinnati.

3. DESCRIPTION OF THE CINCINNATI URBAN RUNOFF MODEL

The Cincinnati Urban Runoff Model is intended for an urban watershed of at least several hundred acres or more in area; however, it may be applied to any size by appropriate scaling. For the development of the model, a deterministic method with parametric system synthesis has been used. A number of requirements and procedures are prescribed in general form below.

a. The total drainage basin is divided into a number of catchments having an area between 10 and 20 acres; however, bigger catchments can be used with a relative loss in accuracy. Each catchment is then further divided into a number of subareas or subcatchments. Subcatchments should be areas with uniform slope and ground cover. The subdivision of each catchment into subcatchments can be done accurately (i.e. the roof of a house can be represented by two subcatchments of different slope) or can be done approximately (considering larger areas as subcatchments).

b. There is a wide flexibility in dividing a catchment into subcatchments for runoff computations. The maximum number of subcatchments is limited by the computer storage availability and by the large amounts of computer time needed to execute a program with many subcatchments. Also, while the subdivision described can be taken to infinitesimal detail in theory, manpower requirements become prohibitive in practice. Sensitivity analyses with coarser subdivisions have indicated substantially the same answers at savings in time and cost [9].

c. Street gutters are considered to conduct the overland flow from the subcatchments to the inlets and catch basins of the sewer system. From there the flow is routed through the sewer pipe system of the catchment to the junction with the main sewer of the drainage basin.

d. This approach is followed sequentially for each catchment and finally the discharges from each catchment are routed through the main sewer to the sewer outlet of the entire watershed.

e. The Cincinnati Urban Runoff Model can employ a different rainfall pattern over each catchment if a sufficient number of rain gages covers the watershed defining the different rainfall patterns. However, the rainfall pattern has to be considered uniform over each catchment. This is reasonable because of the relatively small area of the catchments.

f. The developed computer program accepts as input any rainfall in the form of an hyetograph curve. However, in order to select a design storm pattern, consideration should be given to the fact that high-intensity, short-duration rainfall is normally the main, if not sole, type of precipitation contributing to critical runoff rates. This type of rainfall is usually associated with thunderstorms. Intensity-duration-frequency data of such rainfall in the United States were first analyzed by Yarnell [34], but are now being modified and improved with additional data by the U.S. Weather Bureau [30].

g. It is considered that infiltration in pervious subcatchments follows Horton's equation [10], suitably modified if the initial rainfall intensity is less than the initial infiltration capacity. Infiltration capacity is estimated from the "infiltration capacity curves" of Jens [12] if infiltration data are not available from the basin under consideration.

h. An equation by Linsley [14] has been used as the basis for the derivation of the equation for the rate of depression storage supply.

i. By subtracting the actual infiltration rate and the depression storage supply from the rainfall intensity, the rainfall excess which produces the overland flow is found.

j. Differential equations of continuity and momentum are used to compute the overland flow in connection with an empirical relationship proposed by Crawford [8]. Considering that at least one subcatchment feeds a gutter, the lateral inflow to the gutters due to the overland flow can be obtained. The flow into the gutters is unsteady spatially varied. Simplifying the continuity and momentum equations, there results the inflow hydrograph, i.e. the discharge versus time entering to each catch basin.

k. For flow inside the sewer system, a method of routing through the sewers by time-offset is used because of its simplicity and accuracy. However, the sewer outlet must be unsubmerged and free flowing without backwater effects. If backwater effects exist, the routing method must be modified accordingly.

Summarizing, the proposed mathematical model requires a rainfall hyetograph and data from topographic maps and/or aerial photographs; it simulates infiltration, surface retention, overland flow, gutter flow, and calculates the hydrographs of the flow inside the sewer system of the urban area under consideration. The model can be used to improve existing sewer systems or to design new ones before urban development takes place.

4. INFILTRATION

It has generally been found that during continuous rain the infiltration capacity of a given terrain decreases at first rapidly, then approaches an asymptotic line giving a constant infiltration capacity (commonly 1.5 or 2 hours after the beginning of the rain).

During infiltration into a soil surface, the rate of change of the infiltration capacity can be assumed proportional to the difference between the present and ultimate infiltration capacity. In the equation below, f is the present and f_c the ultimate value of the infiltration capacity.

Considering proportionality:

$$- \frac{df}{dt} = K\ (f - f_c) \quad \text{where K is a positive constant}$$

$$- \frac{df}{(f - f_c)} = K\ dt$$

$$d \ln (f - f_c) = - K\ dt$$

$$\ln (f - f_c) = - Kt_f + C$$

When $t_f = 0$ then $f = f_o$, therefore $C = \ln (f_o - f_c)$

then: $$\ln \frac{f - f_c}{f_o - f_c} = - K\ t_f$$

or $$f - f_c = (f_o - f_c)\ e^{-K\ t_f}$$

so $$f = f_c + (f_o - f_c)\ e^{-K\ t_f} \tag{1}$$

Equation (1) is Horton's equation of infiltration capacity [10], where:

f = infiltration capacity in (in/hr)

t_f = time in minutes measured from the beginning of the infiltration capacity curve

f_c = the constant rate at which f is approached asymptotically as time continues

f_o = infiltration capacity when $t_f = 0$

K = decay rate of infiltration which is a positive constant, with per minute units

The main advantage of equation (1) is that f_o, f_c, K may be related to physical characteristics of the soil types for which curves are available, and this may therefore eventually make possible predictions of the infiltration capacity of soil types for which no experimental data have been obtained [11].

For present purposes streets and mechanically compacted soil will be considered as having negligible infiltration capacity.

If there are no data available concerning infiltration capacity, K, f_c, and f_o can be evaluated from S.W. Jens' infiltration capacity curves (Figure 134) which have been made for turfed areas of airfields and have been used widely for city lawns [12]. Evaluations of infiltration capacity for different soils and cover classifications can be found also in the Soil Conservation Service Handbook [13] and in the ASCE Hydrology Handbook [11].

Curve (a) is representative of generally wet antecedent conditions and heavy rainfall within the preceding 24 hours. Curve (b) is representative of normal antecedent conditions with no considerable application of precipitation for the preceding three days.

Infiltration capacity values shown in Figure 134 can be accepted as reasonably representative of the values for a turfed cover for a rather wide range of soils [12].

The constant K in equation (1) can be determined if f_c and f_o are known by plotting $(f - f_c) = (f_o - f_c)e^{-Kt_f}$ in semilogarithmic paper for an observed series of values of f and t_f. Then the plot will be a straight line with slope K.

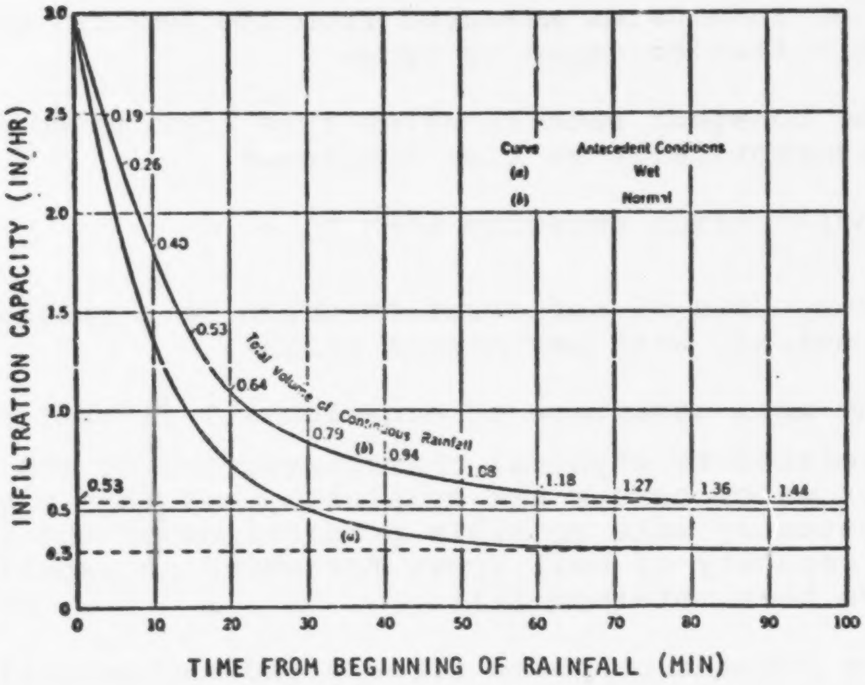


Figure 134. INFILTRATION CAPACITY CURVES FOR WET AND NORMAL ANTECEDENT CONDITIONS OF TURF AREAS [12]

Considering normal antecedent conditions of turf areas (curve b):

$f_c = 0.53$ in/hr

$f_o = 3.00$ in/hr

$K = 0.0697$ 1/min

Considering wet antecedent conditions of turf areas (curve a):

$f_c = 0.30$ in/hr

$f_o = 3.00$ in/hr

$K = 0.0894$ 1/min

The area under the infiltration capacity curve represents the accumulated mass infiltrated, which is:

$$F = \int_0^{t_f} \frac{f}{60}\, dt$$

or
$$F = \frac{f_c}{60} t_f + \frac{(f_o - f_c)}{60\ K} \left[1 - e^{-Kt_f}\right] \qquad (2)$$

Where F is the total inches infiltrated up to the time t_f.

For normal antecedent conditions in turf areas, equations (1) and (2) become:

$$f = 0.53 + 2.47\ e^{-0.0697t_f}$$

$$F = 0.00883\ t_f + 0.59 \left[1 - e^{-0.0697t_f}\right]$$

If the rain starts and continues with an intensity i greater than f, then time t_f in all the above mentioned equations is also the time t in minutes from the beginning of the rainfall. Therefore, the infiltration capacity curve and the rainfall intensity curve start together (Figure 135).

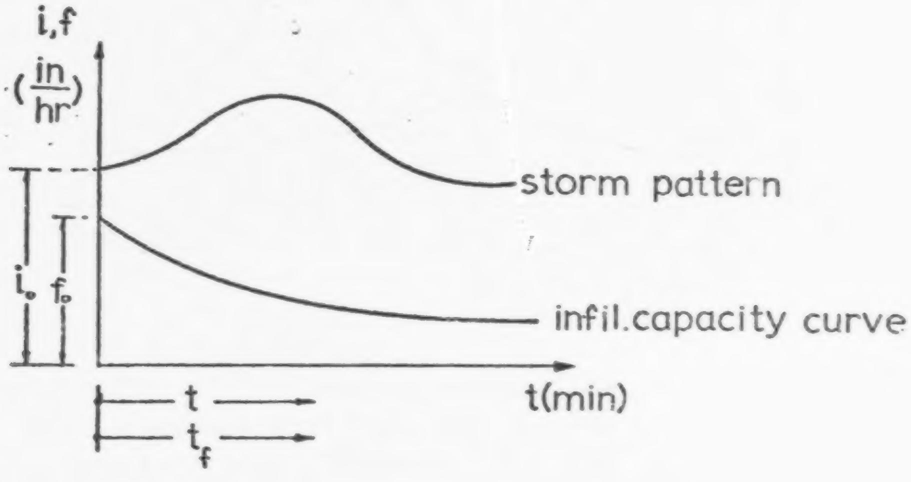
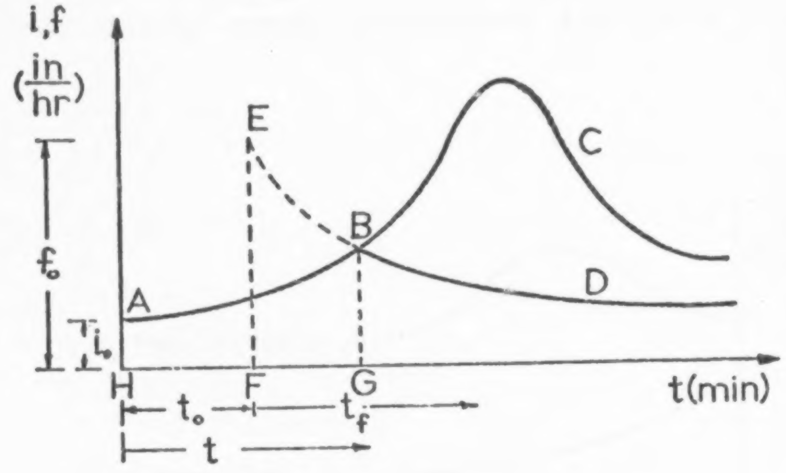


Figure 135. STORM PATTERN AND INFILTRATION CAPACITY IN CASE $i_o > f_o$

But if at the beginning of the rainfall the rain intensity rate is less than the infiltration capacity, then the actual infiltration rate is less than the infiltration capacity and the infiltration capacity curve will be offset by an amount of time t_o. Then t_f in equations (1) and (2) is the time from the beginning of the infiltration capacity curve, and the corresponding time t from the beginning of the rain now becomes $t = t_o + t_f$.

When the rain starts, the actual infiltration curve follows the rainfall intensity curve until it intercepts the offsetted infiltration capacity curve at a point B, following which it proceeds along the latter, as shown in Figure 136.

Assuming that overland flow and depression storage supply commences when the antecedent mass of rainfall equals the antecedent mass of infiltration, the offset time t_o can be defined as the time necessary for (area ABGH) = (area EBGF). If the intensity of the rain is known at all instants, t_o can be found as just explained.



ABC	storm pattern
ABD	actual infiltration rate
EBD	infiltration capacity curve

Figure 136. STORM PATTERN AND INFILTRATION CAPACITY IN CASE $i_o < f_o$

The critical time t_{fc} from the beginning of the rainfall excess required for the infiltration capacity f to drop from its initial value f_o to its asymptotic constant value f_c (i.e. to drop from f_o to approximately 1.01 f_c) is:

$$t_{fc} = \frac{1}{K} \ln \frac{100 (f_o - f_c)}{f_c}$$

and usually varies from 1.5 to 2 hours.

Analytical Method to Find the Offset of the Infiltration Capacity Curve by Computer, when $i_o < f_o$

Suppose that the intersection of the rainfall intensity curve and the shifted infiltration capacity curve takes place after a time $(t_o + t)$ from the beginning of the rainfall and after a time t from the beginning of the infiltration capacity curve. A computer method is presented, with the following details.

The duration of the rainfall has been divided into n small increments DT. Increments of 5 minutes are usually sufficient. However, the computer program can run with time increments of any value. The time from the beginning of the infiltration capacity curve has been divided into increments of the same magnitude.

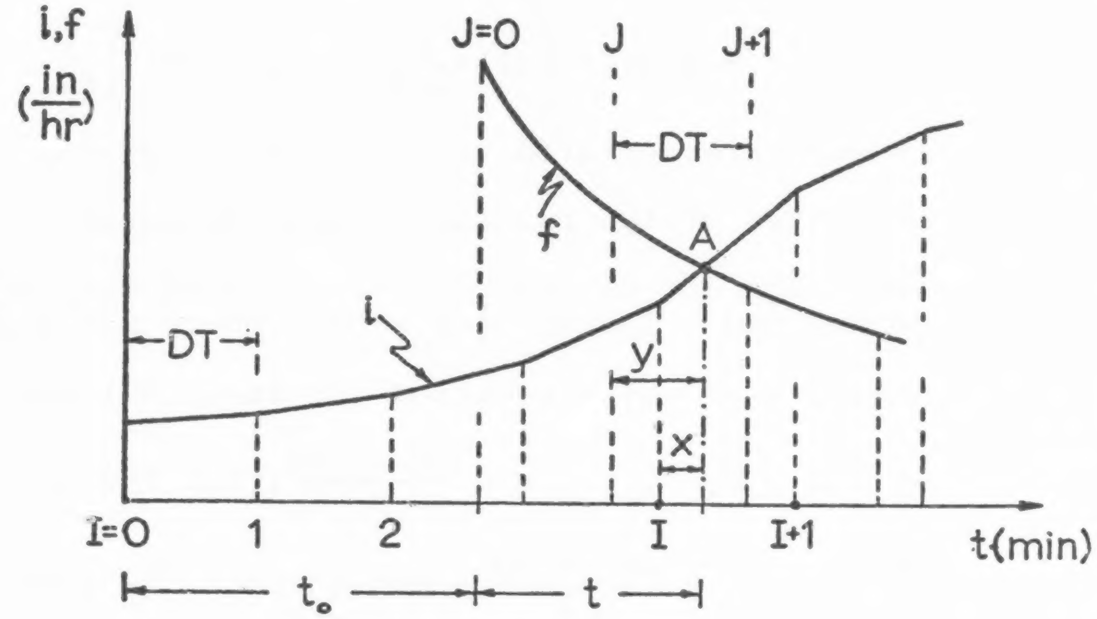


Figure 137. OFFSET OF INFILTRATION CAPACITY CURVE

It is reasonable to assume that the segments of duration DT of the rainfall hyetograph are straight lines. The accuracy of this assumption is greater when the increments of time DT become smaller.

Two conditions have to be satisfied at the point of the intersection A:

(a) rainfall intensity at A = infiltration rate at A

(b) accumulated mass precipitated until A = accumulated mass infiltrated until A.

From the condition (a):

$$f_c + (f_o - f_c)\, e^{-Kt} = i(I) + \frac{x}{DT}\,[i(I+1) - i(I)]$$

$$\text{or} \quad e^{-Kt} = \frac{i(I) + \frac{x}{DT}\,[i(I+1) - i(I)] - f_c}{f_o - f_c}$$

$$\text{so} \quad t = -\frac{1}{K}\ln\left[\frac{i(I) + \frac{x}{DT}\,[i(I+1) - i(I)] - f_c}{f_o - f_c}\right] \tag{3}$$

if and only if the argument of the logarithm is between 0.0 and 1.0.

From the condition (b):

$$\frac{f_c}{60}\,t + \frac{f_o - f_c}{60K}\,(1 - e^{-Kt}) = mi(I) + \left[i(I) + \frac{x}{2DT}[i(I+1) - i(I)]\right]\frac{x}{60} \tag{4}$$

where mi(I) = mass precipitated until a time (I·DT)

and i(I) = ordinates of the rainfall intensity curve.

Each time increment DT can be divided in as many n increments as desired. Therefore x = DT/n where n = 1,2,..

The computer will solve equation (3) for I = 1 and

$x = 0,\ x = \frac{DT}{n},\ x = \frac{2DT}{n},\ \ldots\ x = \frac{(n-1)\ DT}{n}$; for I=2 and

$x = 0,\ x = \frac{DT}{n},\ \ldots\ x = \frac{(n-1)\ DT}{n}$; and so on. The time t which will be found for each combination of I and x will be substituted (together with the corresponding values of I and x) into equation (4).

When equation (4) is satisfied, then the computation will stop and the values of t, I, and x will be stored. The computation of the actual infiltration curve will follow, and the stored value of t will be used to start the calculation of the surface retention.

5. SURFACE RETENTION

Surface retention includes interception, depression storage, and evaporation during a storm.

Interception by vegetation may range from 0.01 to 0.5 inches in forest areas, but is not usually significant in urban drainage.

Evaporation is of little significance for the short rainfall duration encountered in urban storm drainage design.

Therefore, for simplicity of the model, interception and evaporation will be assumed negligible for an urban area.

Some investigators have assumed that all depressions must be filled before overland flow begins. The real situation is that almost immediately after the beginning of the rainfall excess, the smallest depressions become filled and overland flow begins. Most of this water in turn fills larger depressions, but some of it follows an unobstructed path to the collecting gutters.

Linsley, Kohler, and Paulhus [14] recommended an exponential relationship

$$V_s = S_d \left[1 - e^{-N(P - F)} \right] \tag{5}$$

where V_s = volume of water in depression storage at any time throughout a storm, in inches

S_d = total depression storage capacity of the basin, in inches

P = accumulated volume of water precipitated in inches

F = accumulated volume of water infiltrated in inches

N = storage constant

Assuming that the initial increment of rainfall excess is completely retained by depressions:

$$\frac{dV_s}{d(P - F)} = 1 \quad \text{when } (P - F) \text{ is near zero.}$$

Then by differentiation can be found that

$$N = \frac{1}{S_d} \qquad (6)$$

If the depression storage supply is s (in/hr), then:

$$\int_0^t s\, dt = V_s$$

where t is time in hours from the beginning of the rainfall excess.

$$\text{Then} \qquad s = \frac{d}{dt}\left[S_d - S_d\, e^{-\frac{P - F}{S_d}} \right]$$

$$\text{or} \qquad s = \frac{d}{dt}\left[(P - F)\, e^{-\frac{P - F}{S_d}} \right]$$

$$\text{but} \qquad \frac{d}{dt}(P - F) = (i - f) \qquad \text{(in/hr)}$$

$$\text{therefore} \qquad s = (i - f)\, e^{-\frac{P - F}{S_d}} \qquad (7)$$

Of course on impervious areas where infiltration is zero and also accumulated volume of water infiltrated does not exist, equations (5) and (7) become

$$V_s = S_d \left[1 - e^{-\frac{P}{S_d}} \right]$$

$$s = i\, e^{-\frac{P}{S_d}}$$

Horton [15] stated that the depression storage "commonly ranges from 0.125 to 0.75 inches for flat areas and from 0.5 to 1.5 inches for cultivated fields and for natural grass lands or forests". On moderate or gentle slopes he estimated that pervious surface depressions "can commonly hold the equivalent of 0.25 to 0.5 inch depth of water and even more on natural meadow and forest land".

Hicks [16] has used depression storage losses of 0.20 inch for sand, 0.15 inch for loam, and 0.10 inch for clay, considering high rates of rainfall and runoff.

Recent gagings show depression storage to be about 0.3 inch in forest litter, 0.2 inch in good pasture, and 0.05 to 0.10 inch in smooth cultivated land [17].

If there are no data available, the overall depression storage capacity can be assumed equal to 0.25 inch on pervious urban areas which usually present individual depressions with depths from 0 to 0.5 inches. The overall depression storage on impervious urban areas can be assumed equal to 0.0625 inch with the depth of depressions ranging from 0 to 0.125 inches. These assumptions, when there is lack of data about the basin under consideration, seem reasonable from observations made during rainfalls [4], [9], [18].

A typical computer output of the developed computer program (Appendix III) simulating infiltration and depression storage supply is presented graphically in Figure 138.

Subtracting the calculated values of infiltration and depression storage supply from the rainfall intensity at any time, the rainfall excess which produces the overland flow is obtained.

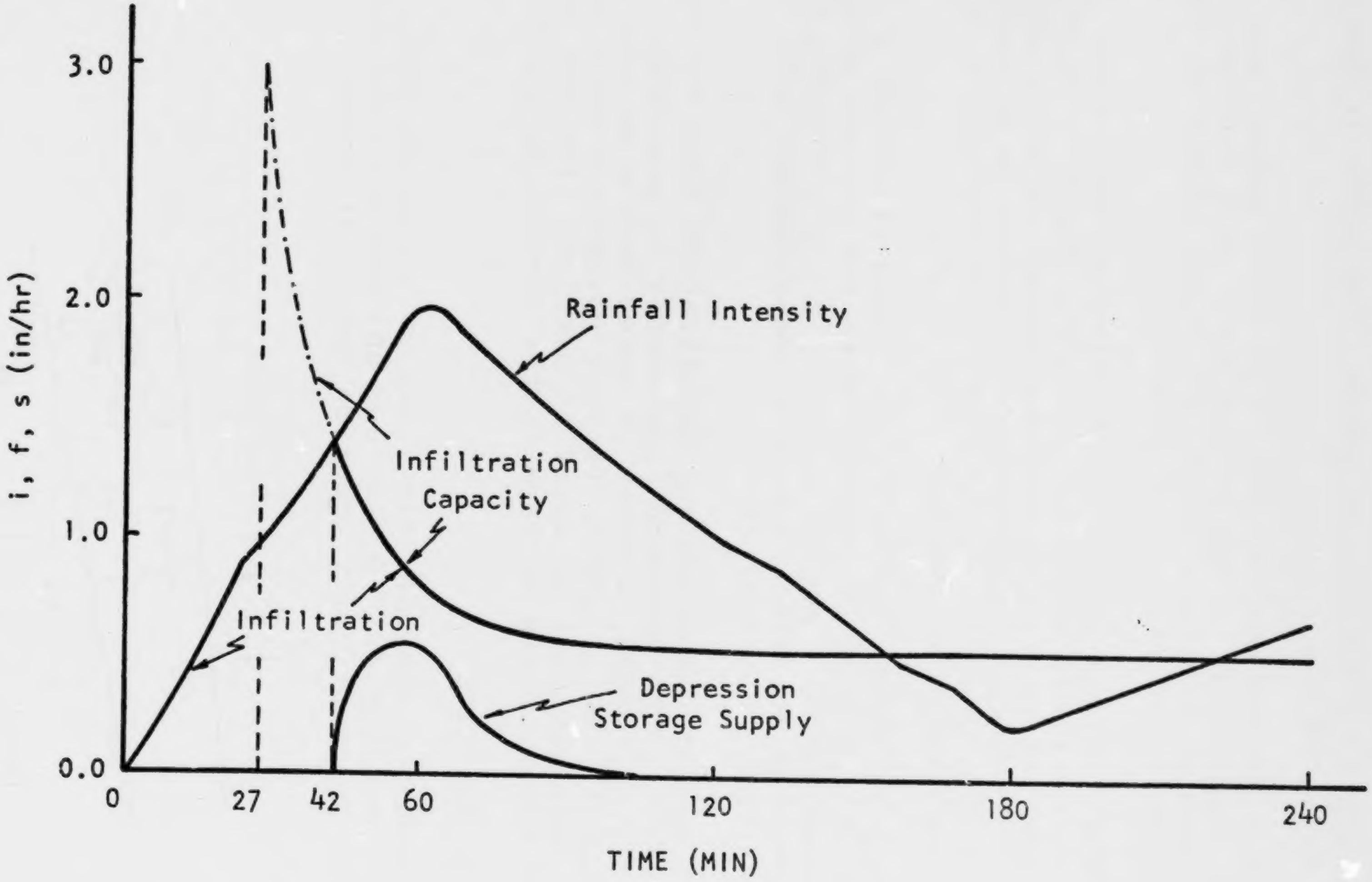


Figure 138. SIMULATION OF INFILTRATION AND DEPRESSION STORAGE SUPPLY

6. OVERLAND FLOW

En route to a channel or gutter, the water is designated as overland flow.

Because of the complex nature of flow, models are generally simplified to two dimensions. This was the approach used by Keulegan (1944) who first worked on the spatially varied unsteady flow problem.

Horton [19] and Izzard [3] studied unsteady flow across sloping plane surfaces. However, Horton's equation for overland flow has limited experimental support, and Izzard's dimensionless hydrograph is limited to situations where $iL < 500$, where i is the rainfall intensity in in/hr and L is the length of overland flow in ft.

In the following the build up of the flow over a sloping plane is treated starting with the partial differential equations of momentum and continuity governing the two-dimensional overland flow on a plane surface with vertical inflow (rainfall excess).

Subtracting retention and infiltration from the rainfall intensity, the overland flow supply is obtained. As this supply increases, a sheet of water builds up over the surface and the water starts to flow toward the collecting gutter. This movement of water is overland flow and the volume of water on the surface is the surface detention

Continuity Equation

Considering flow of unit width on a surface with small bottom slope ($\sin\theta \cong \theta$) and uniform velocity distribution, the continuity equation can be written:

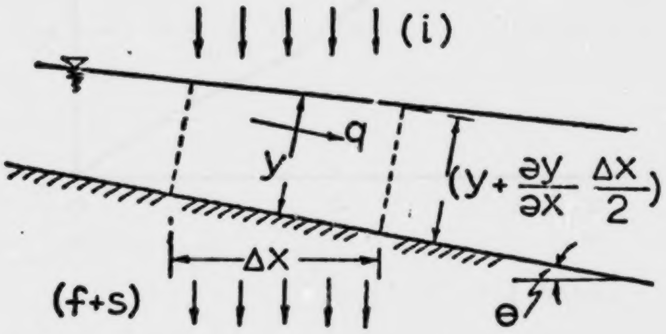


Figure 139. OVERLAND FLOW

$$\frac{\partial q}{\partial x}\,\Delta x + \frac{\partial A}{\partial t}\,\Delta x = i\,\Delta x - f\,\Delta x - s\,\Delta x$$

or
$$\frac{\partial q}{\partial x} + \frac{\partial y}{\partial t} = i - f - s$$

because Area $A = y \cdot 1$, where y is the depth of flow in ft, q is the discharge in cfs/ft of width, and $(i - f - s)$ is the inflow or supply rate in cfs/sq.ft.

Momentum Equation

The momentum equation can be written in the following form, neglecting the raindrop momentum due to the impact of the raindrops on the surface

$$\frac{\partial V}{\partial t} + V\,\frac{\partial V}{\partial x} + g\,\frac{\partial y}{\partial x} = (i-f-s)\,\frac{V}{y} - \left(1+\frac{2y}{b}\right)\frac{\tau_0}{\rho y} + g\,\sin\theta \quad (9)$$

where V is the velocity of flow

τ_0 is the shear stress at the bottom

ρ is the water density

b is the width of flow perpendicular to the direction of flow. For overland flow $b \gg y$, so $\frac{y}{b} \cong 0$.

In comparison with gravity, the rainfall-infiltration process has a negligible effect upon the flow dynamics [20]. Therefore the term $(i - f - s)$ can be omitted from the momentum equation.

An order of magnitude analysis for overland flow can lead to the conclusion that free surface slope and inertia terms in equation (9) are negligible in comparison with those of bottom slope and friction.

Equation (9) then reduces to the well-known relation for steady uniform flow in a wide channel:

$$\tau_0 = \gamma\ y\ \sin\theta \tag{10}$$

where γ is the specific weight of water.

Laminar Flow

For laminar flow Newton's law of viscosity applies:

$$\tau = \mu \frac{dv}{dy'} \tag{11}$$

relating the dynamic viscosity μ and the shear stress τ at a distance y' from the bottom. Equation (10) can be written for any laminar layer:

$$\tau = \gamma\ (y - y')\ \sin\theta$$

$$\text{or} \quad \tau = \frac{g\ \mu}{\nu}\ (y - y')\ \sin\theta \tag{12}$$

where ν is the kinematic viscosity in (ft^2/sec).

Then from equations (11) and (12):

$$dv = g\ \frac{\sin\theta}{\nu}\ (y - y')\ dy'$$

$$v = \frac{g\sin\theta}{\nu}\ (yy' - \frac{y'^2}{2})$$

Therefore the average velocity V will be:

$$V = \frac{1}{y} \int_0^y v dy' = \frac{g\sin\theta}{\nu}\ \frac{y^2}{3}$$

$$\text{so} \quad q = \frac{g\sin\theta}{3\nu}\ y^3 \tag{13}$$

Turbulent Flow

For turbulent flow, Manning's equation can be used, considering the hydraulic radius R equal to the depth y.

$$q = 1.49 \frac{\sqrt{\sin\theta}}{n} y^{5/3} \tag{14}$$

with q (cfs/ft width) and y (ft).

Equations (13) and (14) are of the general form

$$q = K y^{m} \tag{15}$$

(a) for laminar flow:

$$m = 3 \quad \text{and} \quad K = \frac{g\sin\theta}{3\nu} \left[\frac{1}{\text{sec.ft}}\right]$$

with g in (ft/sec^2) and ν in (ft^2/sec)

(b) for turbulent flow:

$$m = \frac{5}{3} \quad \text{and} \quad K = 1.49 \frac{\sqrt{\sin\theta}}{n} \left[\frac{\text{ft}^{1/3}}{\text{sec}}\right]$$

Equation (15) is a good approximation for discharge of surface flow for practical purposes, supported by Keulegan [21], Chow [22], and Eagleson [20].

Criteria of Flow Regime

Horton [23] believed that the Reynolds criterion is not satisfactory for sheet flow over relatively rough surfaces. He found that the point of equal velocities represents the minimum amount of energy capable of maintaining turbulent flow.

Equating the velocities for laminar and turbulent flow, the following relationship is obtained:

$$\frac{g\sin\theta}{\nu} \frac{y^2}{3} = \frac{1.49}{n} y^{2/3} \sqrt{\sin\theta}$$

$$\text{or} \quad y^{2/3} \sqrt{\sin\theta} = \frac{4.47 \nu}{32.17 n y^{2/3}}$$

Substituting the above in Manning's equation we get:

$$V = \frac{\nu}{4.83\ n^2\ y^{2/3}} \qquad (16)$$

The flow cannot be turbulent if the velocity is less than that given by equation (16).

Therefore because $q = Vy$, turbulent flow occurs if:

$$q < \frac{\nu\ y^{1/3}}{4.83\ n^2} \qquad (17)$$

otherwise the flow is laminar.

Izzard (1946) believed the overland flow to be turbulent for:

$$i\ L > 500$$

where i = supply rate (in/hr)

L = length of overland flow (ft)

But the experimental data of Izzard and Augustine and of Ekern (1950) support the hypothesis that rainfall causes the flow to be turbulent even when $i\ L \ll 500$.

Field investigations have shown that overland flow in thin sheets may be entirely turbulent, or partly turbulent and partly laminar. For smooth surfaces, it appears that laminar flow exists with the possibility of changing from laminar to turbulent, and vice versa, within a short distance. For subdivided flow through grass, most of the experimental data indicate that the flow is laminar to some extent and may be represented by a condition of 75% turbulent flow, in which case the profile of overland flow can be assumed parabolic [24] with an equation like (15), having an $m \cong 2$.

Also experimental measurements of surface detention show that high intensity rainfalls often yield Reynolds numbers that indicate turbulent flows.

However, because the division between the laminar and turbulent ranges is difficult to establish and because of the nature of urban areas, the turbulent range equations have been finally selected in the development of the

Cincinnati Urban Runoff Model.

Overland Flow Equations

Overland flow can be defined by the system:

$$\frac{\partial y}{\partial t} + \frac{\partial q}{\partial x} = i - f - s \tag{8}$$

$$q = Ky^m \tag{15}$$

These equations constitute a Kinematic wave problem which has been solved by Eagleson [20] for $s = 0$, i and f constants in time and space, using the method of characteristics.

Here, an attempt has been made to solve equations (8) and (15) considering rainfall intensity, infiltration, and depression storage supply as variables of time.

The only rigorous general methods for simulating unsteady overland flow are finite difference techniques for the numerical solution of the governing partial differential equations. However, the accuracy to be gained by using so laborious methods for overland flow is still subject to question because of the limited accuracy of the basic field data.

Therefore an analytical method based finally on an empirical relationship between outflow depth and detention storage [8] has been chosen in connection with a routing equation.

From the continuity equation (8), at equilibrium:

$$\frac{\partial y}{\partial t} = 0$$

$$\text{and } q = rx \tag{18}$$

where $r = (i - f - s)$ is the overland flow supply rate in (cfs/ft^2) and q is the discharge at equilibrium in cfs/ft of width at any section x on the flow plane. Therefore the outflow at equilibrium is $q_e = rL$.

Also as Wolf [25] stated, the change in discharge as a function of x on a uniformly sloping plane must be zero, before local equilibrium is reached. Therefore the depth at any point on the plane prior to local equilibrium (Figure 140) is

$$y = rt \tag{19}$$

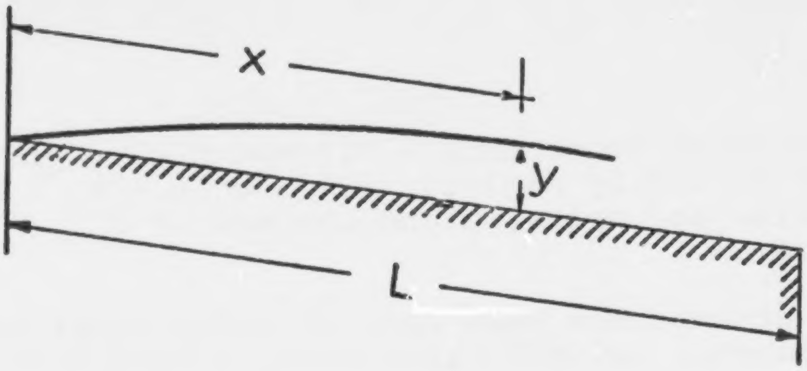


Figure 140. OVERLAND FLOW ON A UNIFORMLY SLOPING PLANE

Between time $t = 0$ and $t = t_e$ (where t_e = time to equilibrium in sec), the total inflow will be equal to the total runoff (outflow) plus the surface detention (storage) in units ft^3/ft of width.

The total inflow during this time will be:

$$I = r\,L\,t_e \tag{20}$$

The surface detention will be:

$$D_e = \int_0^L y\,dx$$

but equation (15) gives $y = \left(\frac{q}{K}\right)^{1/m}$ and also the discharge at equilibrium from equation (18) is $q = rx$

$$\text{So } D_e = \left(\frac{r}{K}\right)^{1/m} \int_0^L x^{1/m}\,dx$$

$$\text{or } D_e = \left(\frac{m}{m+1}\right)\left(\frac{r}{K}\right)^{1/m} L^{(1 + \frac{1}{m})} \tag{21}$$

Accepting that the total runoff is some fraction (a) of the total inflow:

$$R = arLt_e \tag{22}$$

$$\text{but also} \quad R = \int_0^{t_e} q\,dt \tag{23}$$

From equation (19), from t = 0 to $t = t_e$, the depth at x = L should be:

$$y = \left(\frac{t}{t_e}\right) y_e$$

Then equation (15) becomes:

$$q = Ky^m = K \left(\frac{t}{t_e}\right)^m {y_e}^m$$

Therefore equating (22) and (23):

$$arLt_e = K \left[\frac{y_e}{t_e}\right]^m \int_0^{t_e} t^m dt$$

$$\text{from where } arL = \frac{K{y_e}^m}{m+1}$$

$$\text{but} \qquad q_e = rL = K{y_e}^m$$

$$\text{so} \qquad a = \frac{1}{m+1} \tag{24}$$

Then the total runoff will be $R = \frac{1}{m+1} rLt_e$ (25)

The necessary time to reach equilibrium now can be found from the equation: $I = D_e + R$ substituting the expressions of equations (20), (21), (25). Therefore:

$$rLt_e = \left(\frac{m}{m+1}\right)\left(\frac{r}{K}\right)^{1/m} L^{(1+1/m)} + \left(\frac{1}{m+1}\right) rLt_e$$

from where

$$t_e = \frac{r^{(1/m - 1)} L^{1/m}}{K^{1/m}} \qquad (26)$$

For turbulent flow has been found $m=\frac{5}{3}$ and $K=1.486\frac{\sqrt{S}}{n}$ where S is the slope in ft/ft equal to the sinθ for small slopes.

Then from equation (24): $a = 0.375$

From equation (21): $D_e = \frac{0.493 \; r^{0.6} \; n^{0.6} \; L^{1.6}}{S^{0.3}}$

From equation (26): $t_e = \frac{0.01315 \; L^{0.6} \; n^{0.6}}{r^{0.4} \; S^{0.3}}$

where: D_e = detention storage at equilibrium in ft^3/ft of width

t_e = time to reach equilibrium in minutes

r = overland flow supply rate in cfs/ft^2

S = slope of the plane in ft/ft

L = length of overland flow in ft

n = Manning's roughness coefficient

Detention storage in inches depth per unit area and overland flow supply rates in inches per hour are more easy to visualize. Therefore changing in these units:

$$D_e = \frac{0.009792 \; r^{0.6} \; n^{0.6} \; L^{0.6}}{S^{0.3}} \qquad (27)$$

$$t_e = \frac{0.94 \; L^{0.6} \; n^{0.6}}{r^{0.4} \; S^{0.3}}$$

where now D_e (in/unit area), t_e (min), r (in/hr), L (ft), S (ft/ft).

The rate of discharge from overland flow based on Manning's equation is:

$$q = \frac{1.486}{n} y^{5/3} S^{1/2}$$

where q in cubic feet per second per foot of width and y the depth in feet at the lower edge of the flow plane.

Rewriting Manning's equation with q in inches per hour per unit area, y in inches, and L in ft:

$$q = \frac{1020.7}{n\ L} y^{5/3} S^{1/2} \qquad (28)$$

The most satisfactory empirical relationship found between outflow depth and detention storage for reproducing experimental hydrographs [8] is:

$$y = D \left[1.0 + 0.6 \left(\frac{D}{D_e}\right)^3 \right] \qquad (29)$$

where y is the outflow depth in inches

D_e is the detention storage required at equilibrium for the current rate of inflow in inches per unit area, calculated from equation (27)

D is the current detention storage in inches per unit area

During recession flow, when D_e is less than D, the ratio D/D_e in equation (29) is assumed to be one.

Then equation (28) becomes

$$q = \frac{1020.7}{n\ L} S^{1/2} D^{5/3} \left[1.0 + 0.6 \left(\frac{D}{D_e}\right)^3 \right]^{5/3} \qquad (30)$$

with units q (in/hr/unit area)

L (ft) and D, D_e (in/unit area)

To determine the overland flow runoff hydrograph, a storage routing procedure is used.

The storage equation can be written:

$$r - q = 60 \frac{dD}{dt}$$

where r is the inflow (overland flow supply, known from the infiltration model), q is the outflow, dD is the change in detention storage, and dt is the time increment in minutes.

This equation can be written in an incremental form form [26]

$$\frac{r_1 + r_2}{2} - \frac{q_1 + q_2}{2} = 60 \frac{D_2 - D_1}{\Delta t}$$

or $$\frac{r_1 + r_2}{2} - \frac{q_1}{2} + 60 \frac{D_1}{\Delta t} = \frac{q_2}{2} + 60 \frac{D_2}{\Delta t} \qquad (31)$$

where the subscripts 1 and 2 indicate values at the beginning and the end of the routing increment Δt.

Combining equation (31) with equation (30) we get:

$$\frac{r_1+r_2}{2} - \frac{q_1}{2}+60\frac{D_1}{\Delta t}=\frac{510.35}{nL}S^{1/2}D_2^{5/3}\left\{1.0+0.6\left[\frac{D_2}{D_{e_2}}\right]^3\right\}^{5/3} +60\frac{D_2}{\Delta t} \qquad (32)$$

where D_e can be found for each corresponding r from equation (27).

Now all the known values are at the left-hand side of equation (32). Starting at the first time increment with $q_1 = 0$, $D_1 = 0$ the value of D_2 is found from equation (32) and then q_2 is found from equation (30). Then the calculated values of q_2, D_2 become q_1 and D_1 for the subsequent period and the above process is repeated giving the hydrograph of the overland flow in inches per hour which can be directly compared to the rainfall rates.

However, in order to calculate the discharge entering into the gutters due to the overland flow supply, it is easier to have available the overland flow in cubic feet per second per foot of width. This can be done by multiplying the values of q found from the routing procedure by a factor, i. e.

$$q\ (\frac{\text{cfs}}{\text{ft width}}) = \frac{L\ (\text{ft})}{43200}\ q\ (\frac{\text{in/hr}}{\text{unit area}})$$

Where L is the length of the overland flow.

This method has the advantage in using the Manning's roughness coefficient for which there are a lot of experimental data available for different ground covers (see Appendix I).

This method agreed satisfactorily with the well known Izzard's method [3], [27]. An example output is presented in Figure 141. Overland flow rates by using Izzard's method are less during the beginning of the rainfall excess and higher near the peak than these calculated by using the Urban Runoff Model. But finally the volume of water which ran off is approximately the same (areas under the overland flow curves). The differences can occur due to the fact that Manning's n = 0.35 may not well correspond to Izzard's C = 0.06.

Several computer runs have shown that the Cincinnati Urban Runoff Model simulating the overland flow, finally comes out with a volume of water, which ran off, approximately equal to the total volume of water which was available for overland flow. Differences of less than 5% proved that the model follows adequately the continuity principle.

A brief sensitivity analysis has been done for the overland flow model which proved ever more its accuracy. An example is presented in Figure 142. For bigger Manning's roughness coefficients, the peaks of overland flow hydrographs tend to be higher and to occur at earlier times. For bigger lengths of overland flow the peaks tend to be lower and to occur at a later time, resulting in a flatter overland flow hydrograph.

The described overland flow model offers simplicity and calculating speed compared with other more exact methods, while attempting to maintain a reasonable approximation to physical behavior.

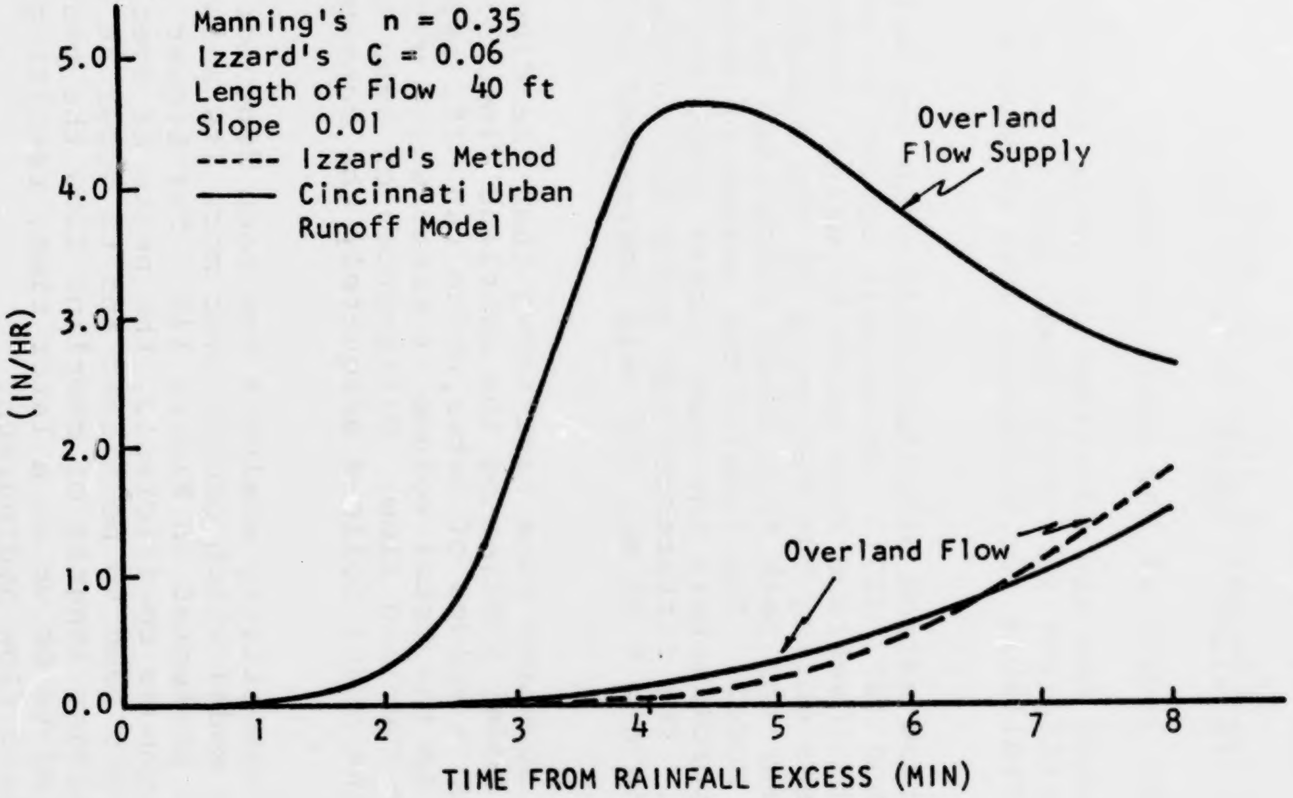


Figure 141. OVERLAND FLOW COMPUTATION BY IZZARD'S METHOD AND BY THE CINCINNATI URBAN RUNOFF MODEL

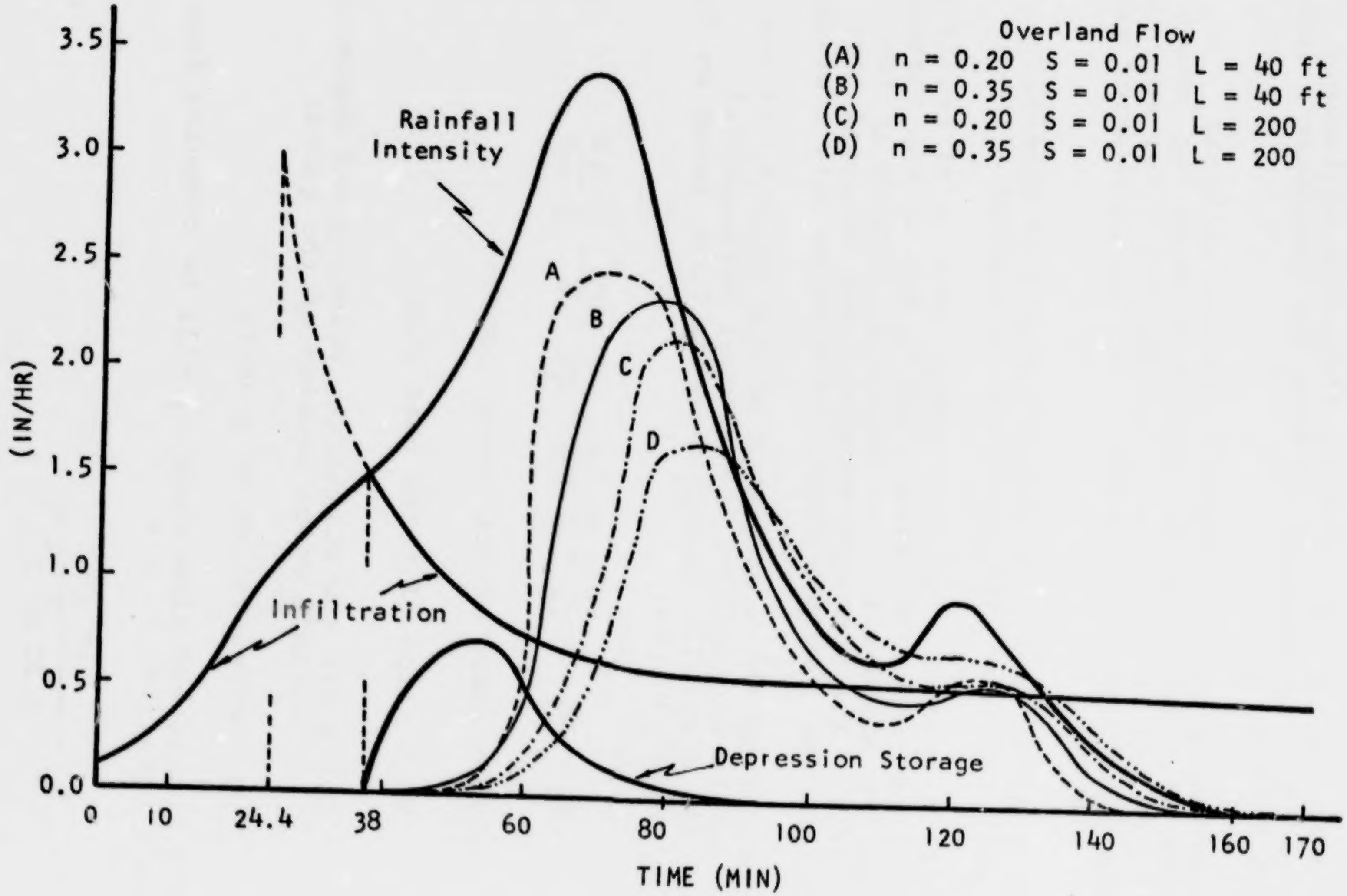


Figure 142.SENSITIVITY OF THE OVERLAND FLOW MODEL

7. GUTTER FLOW

Gutters will gather overland flow in a continuous fashion along their lengths, to feed the catchment's inlets and catch basins.

Experiments by W. I. Hicks, introducing equal increments of flow at regular intervals along the length of a gutter, showed the difference between the water-surface slope and the construction grade. The upstream water slope was much flatter.

The type of flow in roadside gutters is spatially varied unsteady flow with increasing discharge.

In this type of flow, the energy loss is of relatively high magnitude and uncertainty, and is due to the turbulent mixing of the added water and the water flowing in the channel [26]. The momentum equation will be found more correct than the energy equation in solving this problem.

Considering the x component of the velocity of the lateral inflow negligible, the general differential equation for spatially varied unsteady flow, based on the momentum principle, is:

$$S_o - S_f = \frac{\partial y}{\partial x} + \frac{V}{g}\frac{\partial V}{\partial x} + \frac{V}{gA}\frac{\partial Q}{\partial x} + \frac{1}{g}\frac{\partial V}{\partial t} \qquad (33)$$

where S_o = slope of the gutter bottom

S_f = slope of total head line

Q,V,y are the discharge, velocity and depth at a certain cross section of the gutter

g = acceleration of gravity

The total head line slope S_f will be computed from Manning's equation, i. e.:

$$S_f = \frac{Q^2 n^2}{2.22 A^2 R^{4/3}} \qquad (34)$$

where n = Manning's roughness coefficient

A = area of gutter cross section

R = hydraulic radius of gutter cross section

Also $$\frac{V}{g}\frac{\partial V}{\partial x} = \frac{Q}{gA}\frac{\partial}{\partial x}\left(\frac{Q}{A}\right) = \frac{Q}{gA^2}\frac{\partial Q}{\partial x} - \frac{Q^2}{gA^3}\frac{\partial A}{\partial x}$$

therefore $$\frac{V}{g}\frac{\partial V}{\partial x} = \frac{Q}{gA^2}\frac{\partial Q}{\partial x} - \frac{Q^2 T}{gA^3}\frac{\partial y}{\partial x} \qquad (35)$$

where T = width of the water surface.

Also:

$$\frac{V}{gA}\frac{\partial Q}{\partial x} = \frac{Q}{gA^2}\frac{\partial Q}{\partial x} \qquad (36)$$

By using equations (35) and (36), equation (33) becomes:

$$S_o - S_f = \frac{\partial y}{\partial x}\left(1 - \frac{Q^2 T}{gA^3}\right) + 2\frac{Q}{gA^2}\frac{\partial Q}{\partial x} + \frac{1}{g}\frac{\partial V}{\partial t} \qquad (37)$$

The continuity equation for unsteady flow with lateral inflow q_L , is:

$$\frac{\partial Q}{\partial x} + \frac{\partial y}{\partial t} T = q_L \qquad (38)$$

The lateral inflow q_L , available from the overland flow, will be constant along the length of the gutter for a time interval. So q_L is a function of t but not of x.

Because only the discharge entering the sewer catch basins at the downstream end of the gutters is required, equation (37) can be discarded and the Q at the end of the gutters can be obtained by using only equation (38).

For an urban area the lengths of the gutters are relatively small. Also the overland flow supply in cfs/ft is small. Therefore, in a time increment, the change of the depth in the gutter will be very small in comparison with the other terms of equation (38) (see Appendix I).

Omitting the term $\frac{\partial y}{\partial t}$ T equation (38) becomes:

$$\frac{\partial Q}{\partial x} = q_L$$

and integrating: $Q = q_L \cdot L + Q_o$ (39)

where Q_o is the discharge entering the gutter from an upstream gutter, otherwise zero (Figure 143).

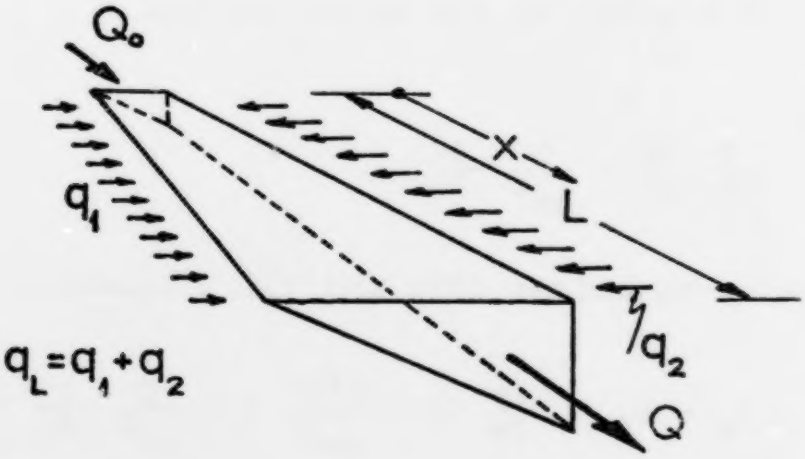


Figure 143. GUTTER FLOW

Once the hydrograph of the discharge entering into an inlet (inflow hydrograph) is obtained from equation (39) it will be routed through the sewer until the next inlet downstream, where a new inflow hydrograph will be added to the routed one and the summation will be routed downstream, and so on.

8. ROUTING THROUGH LATERAL AND MAIN SEWERS

In Chicago, Tholin and Keifer [4] used a time-offset method for conduit routing because of its simplicity compared with other routing methods and because more refined procedures (storage routing) had not led to significantly different results.

Storage routing can be applied if satisfactory discharge-storage relationships are available. This makes necessary the computation of instantaneous backwater curves. Since only the rate of change in storage is necessary to solve the storage equation, it is considered expedient to assume a uniform flow condition for each discharge rate [22] and compute the conduit volume occupied by the flow.

Other methods used in Europe, like the semigraphical method of Hauff and the Italian storage method, are relatively easy to use, but are based on broader assumptions [28].

Generally there are many points of entry (inlets) into a lateral sewer. As points of entry for a main sewer are considered the junctions with the lateral sewers.

Considering a lateral sewer of uniform flow and cross-section, for each inlet a hydrograph of the inflow is available from the gutter flow. It seems more reasonable to use a hydrograph simplified as constant discharge rates for each time increment, rather than a smooth curve discharge-time relationship (Figure 144).

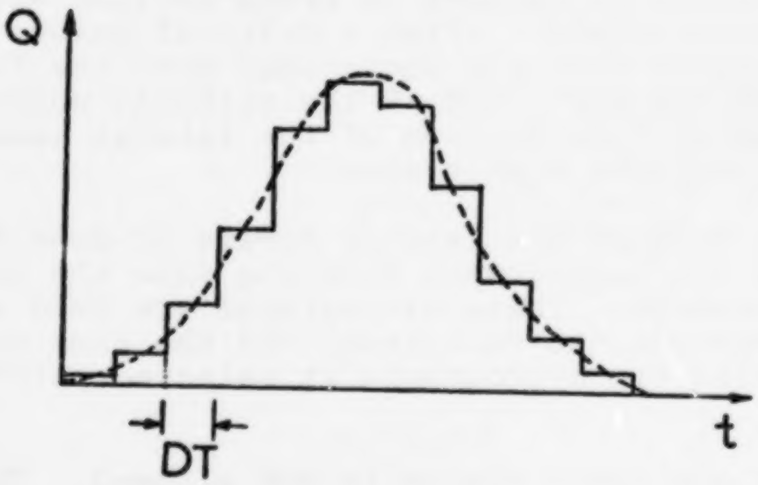


Figure 144. TRANSFORMED HYDROGRAPH

Assuming a uniform flow condition for each discharge rate, Manning's equation can be used to find the depth and the velocity of flow into the sewer for each discharge. So the velocity with which each water volume element is moving downstream will be known.

The average velocity with which the total inflow volume (represented by the area under the hydrograph curve) moves downstream is considered to be the weighted average of all the partial velocities with respect to the corresponding volumes, i. e. multiply each volume element by its velocity, sum the products and divide the sum by the total volume of the water which moves downstream.

Once the average velocity of flow is found and knowing the length of the conduit between inlets, the time needed for the inflow water volume to travel this distance can be found.

Each inflow hydrograph is shifted in time without changing in shape. This has been found satisfactory and convenient, producing only a slightly higher peak rate of flow occurring at a somewhat later time than other more exact methods.

Therefore, for a lateral sewer of uniform slope and cross-section, the computer shifts the first inflow hydrograph the time required to reach the second inlet. Then, it adds the new inlet hydrograph and the shifted one and shifts the summation hydrograph until the next inlet, and so on until it reaches a critical point.

Changes in slope or changes in cross-section are considered as critical points. After a critical point, the procedure is repeated from the beginning, with the first inflow hydrograph the one found at the critical point. This continues until the junction of the lateral sewer under consideration with a main sewer.

The routing through the lateral sewers is done first, in order to find the hydrographs entering into the main sewer at the junctions. These hydrographs are used as inflow hydrographs for the main sewer and the same method is followed to find the hydrographs at selected points of the main sewer.

Overflow of the sewer system is not allowed. The computer compares continuously the maximum capacity of the conduits and the routed discharges. If the capacity of a conduit is not sufficient, the computer indicates that in the output. This justifies the purpose of the present "Model" to design a sewer system or improve an existing one. In case of overflow, the cross-section of the insufficient conduit or its slope or both have to be changed and the computer program rerun.

It is assumed that the Manning's roughness coefficient remains constant as the depth changes (actually the roughness coefficient decreases as the depth of flow increases).

For practical purposes also, it can be assumed that the maximum discharge of a circular conduit does occur at the full depth, because the depth for maximum discharge is so close to the top that there is always a possibility of slight backwater to increase this depth closer or equal to the full depth [22].

Time steps, for the computer solution, may be chosen to coincide with the spacing of the ordinates of the inflow hydrographs for convenience. However, 1 minute intervals will facilitate the addition of the hydrographs.

Let us suppose now that the discharge entering into the sewer during a time increment Δt is Q, taken from the inflow hydrograph. In order to find the velocity with which the volume $(Q \cdot \Delta t)$ is traveling downstream, Manning's equation is used:

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2} \tag{40}$$

where: Q = discharge in cfs

n = roughness coefficient of the conduit (see Appendix I)

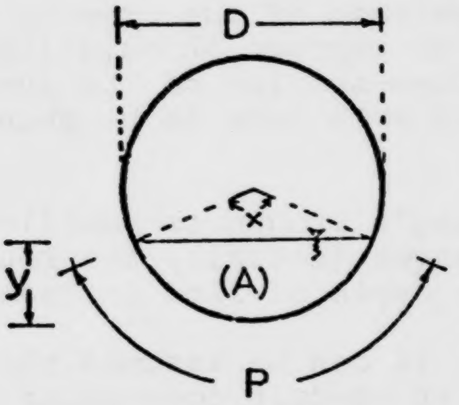
A = cross-sectional area of flow in ft^2

R = hydraulic radius in ft

S = slope of the invert of the conduit in ft/ft

The present method is used for sewers of circular and rectangular cross section. Other shapes can be transformed into these basic two cross sections.

(A) Circular shape:



$$A = \frac{D^2}{8}\,(x - \sin x) \qquad (41)$$

$$P = \frac{D}{2}\,x$$

$$R = \frac{A}{P} = \frac{D}{4}\left(1 - \frac{\sin x}{x}\right)$$

where x in radians and

$$0 \leq x \leq 2\pi$$

The maximum capacity of the pipe is (full flow):

$$Q_{max} = \frac{0.46417}{n}\,D^{8/3}\,S^{1/2}$$

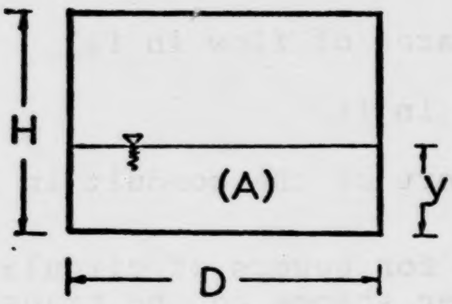
For partially full flow conditions equation (40) becomes:

$$\frac{(x - \sin x)^{5/3}}{x^{2/3}} = \frac{13.566\ Q\ n}{D^{8/3} \cdot S^{1/2}} \qquad (42)$$

The right hand part of equation (42) is known, so equation (42) can be solved by trial to find the angle x. Knowing x, the area of flow A is obtained from equation (41) and the velocity of flow V is obtained from the equation:

$$V = \frac{Q}{A}\ \text{(ft/sec)} \qquad (43)$$

(B) Rectangular shape



$$A = Dy$$

$$P = D + 2y$$

$$R = Dy/(D + 2y)$$

The maximum discharge passing through the conduit (full flow, y = H) is:

$$Q_{max} = \frac{0.9361}{n} \frac{(D\ H)^{5/3}}{(D + H)^{2/3}} S^{1/2}$$

For partially full flow, equation (40) becomes

$$\frac{y^{5/3}}{(D + 2y)^{2/3}} = \frac{Q\ n}{1.486\ S^{1/2}\ D^{5/3}} \qquad (44)$$

The right hand part of equation (44) is known, so can be solved by trial to find the depth of flow y (in ft). Then the velocity of flow will be

$$V = \frac{Q}{Dy}\ \text{(ft/sec)} \qquad (45)$$

If an inflow hydrograph consists of N time increments Δt with the corresponding discharges Q_1 , Q_2 , ..., Q_N , then the average velocity with which the total inflow volume travels downstream until the next inlet is:

$$V_{av} = (V_1Q_1\ \Delta t + V_2Q_2\ \Delta t + \ldots + V_NQ_N\ \Delta t)/(Q_1\ \Delta t + Q_2\ \Delta t + \ldots + Q_N\ \Delta t)$$

or $V_{av} = (V_1Q_1 + V_2Q_2 + \ldots + V_NQ_N)/(Q_1 + Q_2 + \ldots + Q_N)$

where V_1 , V_2 , ... are the velocities with which the volumes $Q_1\ \Delta t$, $Q_2\ \Delta t$, ... travel downstream. These velocities are given from equations (43) or (45).

9. FIRST CASE STUDY ON SIMULATION OF URBAN STORM WATER RUNOFF BY USING THE CINCINNATI URBAN RUNOFF MODEL

A catchment of 10 acres in Chicago, Illinois, has been selected to verify the Cincinnati Urban Runoff Model because detailed data were available and because it is typical of the Chicago street pattern and may be typical of some other cities.

The catchment has been divided into six basic types of subcatchments:

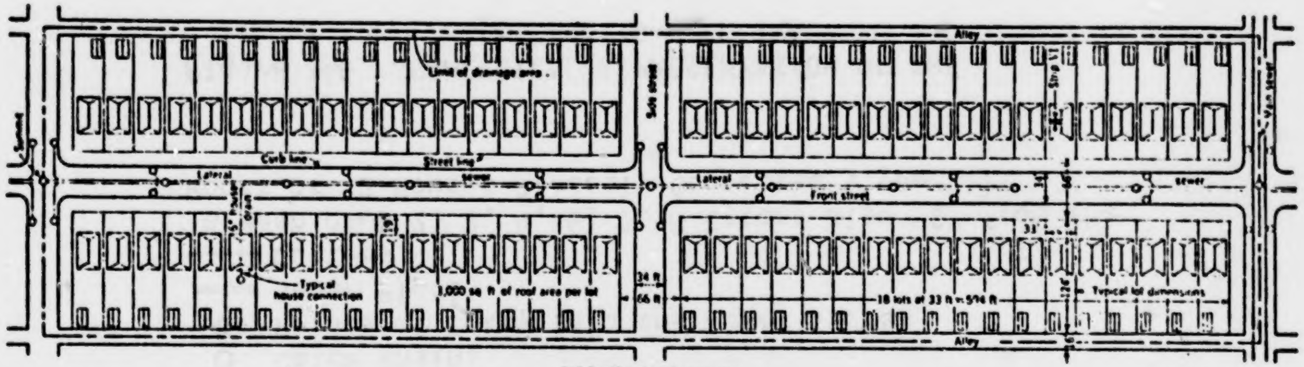
Type	Description	Length (ft)*	Slope*	Manning's n
I	Dense blue grass turf, pervious	16	0.0100	0.350
II	Dense blue grass turf, pervious	40	0.0100	0.350
III	Dense blue grass turf, pervious	70	0.0100	0.350
IV	Street pavement, impervious	17	0.0200	0.013
V	Alley pavement, impervious	8	0.0375	0.013
VI	Common composition roofing, impervious	12	0.6670	0.012

*Parallel to the direction of overland flow

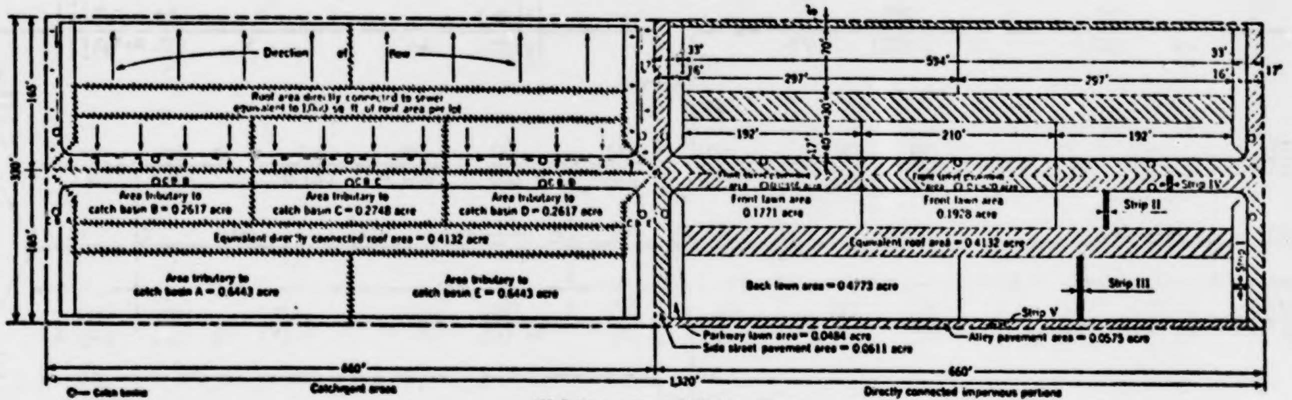
The 10 acres catchment constitutes 5.37 acres (53.7%) of pervious areas and 4.63 acres (46.3%) of impervious areas.

Building roofs are not discharging through the downspouts on the lawn areas, but are connected directly to the underground sewers. To reduce the amount of computations, six roofs were considered directly connected with each catch basin.

The garage roofs, because they discharge onto backyard lawns, were considered a part of the pervious areas.

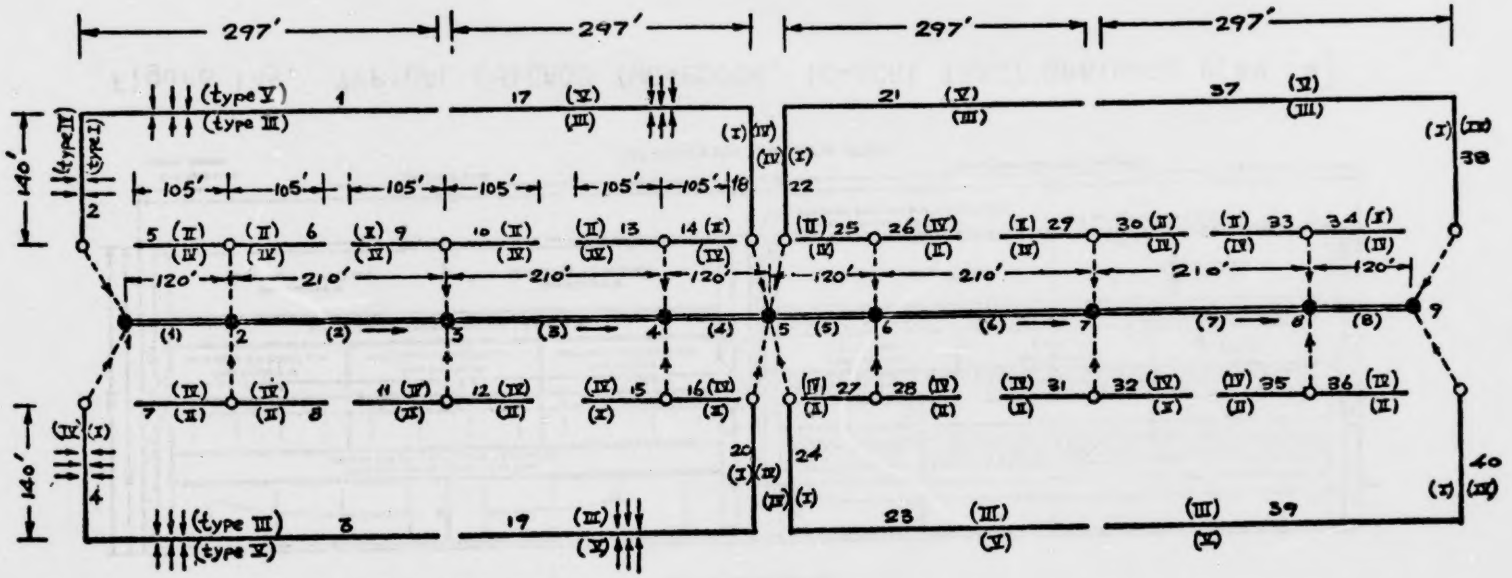

Alley
Limit of drainage area
Side street
Strip VI
Van street
Summit
Curb line
Street line
Lateral
sewer
Lateral
Front street
sewer
5" house drain
Typical house connection
1,000 sq ft of roof area per lot
34 ft
66 ft
33'
124'
18 lots of 33 ft = 594 ft
Typical lot dimensions
Alley

(a) Land use and sewerage


Direction of flow
Roof area directly connected to sewer equivalent to 1,000 sq ft of roof area per lot
C.B. B
C.B. C
C.B. D
C.B. A
C.B. E
Area tributary to catch basin B = 0.2617 acre
Area tributary to catch basin C = 0.2748 acre
Area tributary to catch basin D = 0.2617 acre
Equivalent directly connected roof area = 0.4132 acre
Area tributary to catch basin A = 0.6443 acre
Area tributary to catch basin E = 0.6443 acre
165'
165'
330'
660'
1,320'
Catchment areas
Catch basins
Manholes
33'
16'
17'
594'
297'
297'
70'
30'
40'
17'
192'
210'
192'
33'
16'
17'
Front lawn area 0.1771 acre
Front lawn area 0.1928 acre
Strip II
Strip IV
Equivalent roof area = 0.4132 acre
Strip I
Back lawn area = 0.4773 acre
Strip III
Strip V
Parkway lawn area = 0.0484 acre
Side street pavement area = 0.0611 acre
Alley pavement area = 0.0575 acre
660'
Directly connected impervious portions

(b) Drainage area and direction of flow

Figure 145. TYPICAL CHICAGO TWO-BLOCK, 10-ACRE TRACT DRAINAGE PLAN [4]


297'
297'
297'
297'
140'
140'
105'
120'
210'
(type V)
(type III)
(type I)
(type II)
1
2
3
4
5
6
7
8
9
(1)
(2)
(3)
(4)
(5)
(6)
(7)
(8)
17
18
19
20
21
22
23
24
37
38
39
40

○ Catch Basins
● Points of Entry into the Sewer System
═ Sewer Pipes
— Gutters
In each of ● 2, 3, 4, 6, 7, 8 Enters also "Q" from the Roofs of 12 Houses Directly Connected (Type VI)

Figure 146. GEOMETRICAL REPRESENTATION OF THE CHICAGO 10-ACRE TRACT

The lateral sewer was of circular cross section with a diameter of 3 ft, a slope of 0.0045 ft/ft, and a Manning's roughness coefficient of 0.013.

The subdivision of the catchment into subcatchments, the sewerage, and the directions of overland flow and gutter flow are shown in Figure 145.

Figure 146 represents the catchment as a system of geometric elements, the types of overland flow, gutters, catch basins, and sewer system. The way of numbering the gutters, the points of entry into the sewer system, and the numbering of the pipes is shown on Figure 146.

The computer program for the Cincinnati Urban Runoff Model is inserted in Appendix III. Complete information for the preparation of the data cards is in Appendix IV, and the data cards for the Chicago 10-acre catchment are presented in Appendix V.

A 181-minute design storm was applied to the drainage basin and the computations were made at 5-minute intervals. A detailed computer output for the present case study is available at the Division of Water Resources, Department of Civil Engineering, University of Cincinnati.

For the purpose of comparison, the runoff hydrograph at the outlet of the Chicago 10-acre catchment was calculated by three methods: the Cincinnati Urban Runoff Model, the EPA Storm Water Management Model, and the Chicago Method. The resulted hydrographs are plotted in Figure 147 together with the 181-minute design storm hyetograph which was applied to the drainage basin.

Comments on the Results of the First Case Study

1. The Chicago Method, the EPA Storm Water Management Model, and the Cincinnati Urban Runoff Model, calculated that the peak would occur at about the same time, i.e., 75 minutes from the start of the rainfall as shown in Figure 147.

2. The Chicago Method and the EPA Storm Water Management Model calculated a peak of 18.2 and 22.6 cfs respectively. The Cincinnati Urban Runoff Model calculated a peak of 23.0 cfs, which agreed satisfactorily with the EPA Storm Water Management Model. However, the latter divided the Chicago 10-acre catchment into 80 subcatchments, 40 gutters, and 4 pipes, while the Cincinnati Urban Runoff

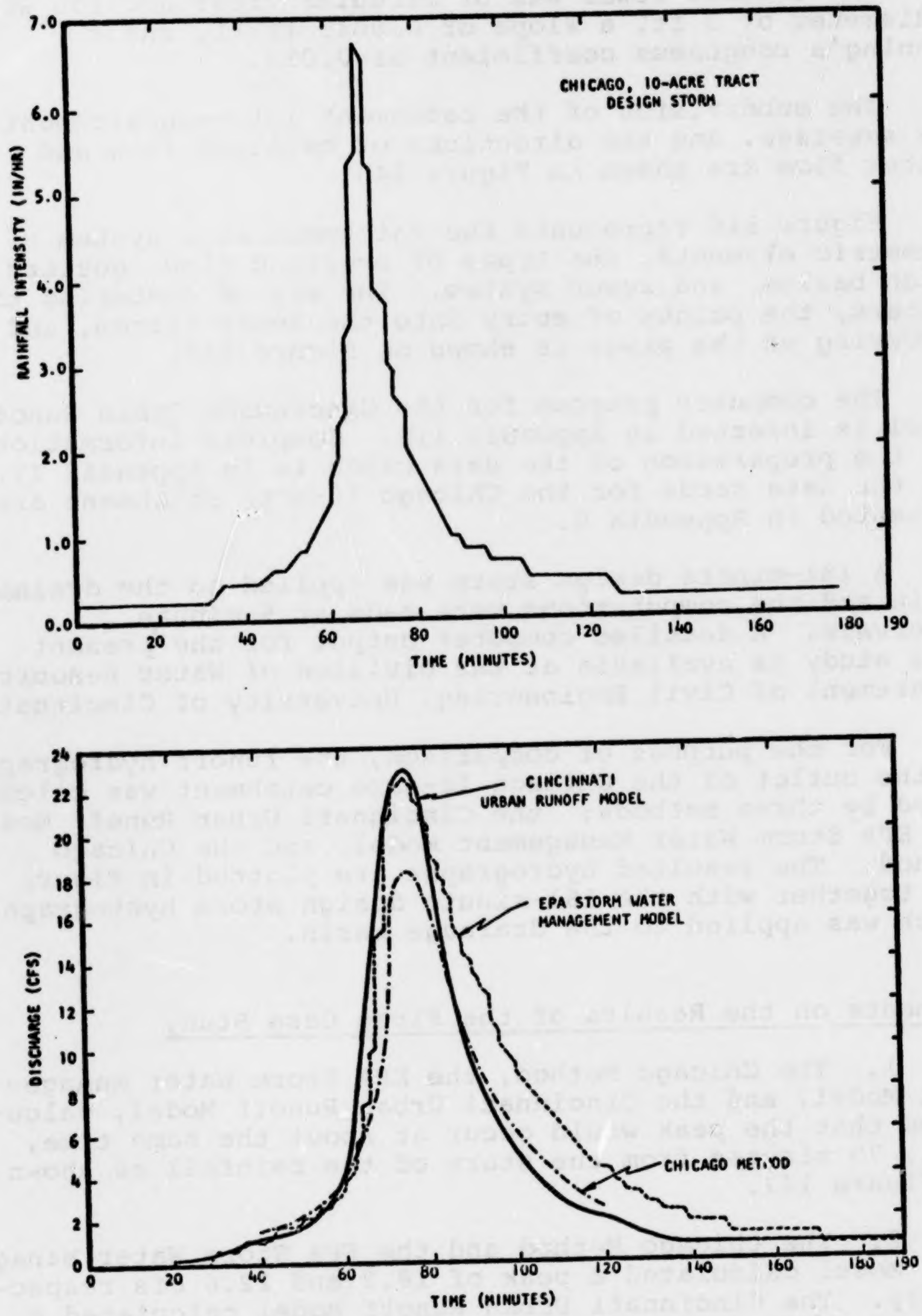


Figure 147. RAINFALL HYETOGRAPH AND CALCULATED RUNOFF HYDROGRAPHS
CHICAGO, 10-ACRE TRACT
DESIGN STORM

Model used repeatedly 6 basic types of subcatchments, 40 gutters, and 8 pipes, therefore reducing the computer time to about 5.5 minutes using the IBM 360/65 computer of the University of Cincinnati.

Comparing the peaks obtained by the three above methods with the Rational Formula $Q = ciA$, the following is obtained:

c = coefficient of runoff equal to 0.40 for relatively flat residential areas, 30% impervious [29]

A = catchment area, 10 acres

i = average rainfall intensity in inches per hour for a duration equal to the time of concentration of the catchment

Q = peak flow rate, in cfs

During the rainfall used in the previously presented study case, 2.50 inches of water precipitated on the catchment. The duration of the rainfall was 181 minutes. This rainfall has a return period of 10 years for the Chicago area [30]. Then the average rainfall intensity, for a time of concentration equal to 15 minutes and a return period of 10 years, is $i = 5$ inches per hour [31]. Therefore, $Q = 0.4 \times 5 \times 10 = 20$ cfs, which agrees satisfactorily with the peaks in Figure 147.

3. The three methods gave a comparable rising limb. However, the Chicago Method and the Cincinnati Urban Runoff Model calculated a lower recession than that obtained from the EPA Storm Water Management Model. This can only be accounted for by a larger infiltration loss. Actually the Chicago Method and the Cincinnati Urban Runoff Model compute the mass curve for infiltration loss. Thus, the infiltration rate tends to be satisfied at any given time even though there is not sufficient rainfall at that moment, under the condition that the accumulated mass precipitated is bigger than the accumulated mass infiltrated.

4. The EPA Storm Water Management Model maintained the mass continuity within 0.1% of the rainfall. No such information was available on the Chicago Method for comparison. The Cincinnati Urban Runoff Mathematical Model maintained the mass continuity within an acceptable 2% of the rainfall as has been shown by a series of case studies performed at the University of Cincinnati. Manpower and computer time did not justify an effort for more accuracy.

10. SECOND CASE STUDY ON SIMULATION OF URBAN STORM WATER RUNOFF BY USING THE CINCINNATI URBAN RUNOFF MODEL

The Oakdale Avenue drainage basin in Chicago, Illinois, has been selected to verify the Cincinnati Urban Runoff Model because rainfall and runoff have been periodically measured and recorded since 1959 by the Chicago Department of Public Works, Bureau of Engineering [32].

The Oakdale Avenue catchment is an urban drainage area 12.9 acres in size (approximately 2 1/2 blocks long by one block wide) and consists entirely of residential dwellings. The catchment has been divided into six basic types of subcatchments:

Type	Description	Length (ft)*	Slope*	Manning's n
I	Grassed, pervious	35	0.010	0.350
II	Grassed, pervious	35	0.010	0.300
III	Grassed, pervious	50	0.010	0.350
IV	Grassed, pervious	23	0.010	0.350
V	Street pavement, impervious	15	0.020	0.013
VI	Common composition roofing, impervious	11	0.667	0.012

*Parallel to the direction of overland flow

The 12.9 acres catchment constitutes 7.05 acres (54.7%) of pervious areas and 5.85 acres (45.3%) of impervious areas.

Building roofs and garage roofs are draining directly into the underground sewers.

The existing 30-inch diameter combined sewer was installed in 1958 and replaced a smaller combined sewer of inadequate storm water capacity. The slopes of the 30-inch lateral sewer vary from 0.0045 upstream to 0.0030 downstream, and the Manning's roughness coefficient is considered to be 0.013.

The plan of the 12.9-acre Oakdale Avenue drainage

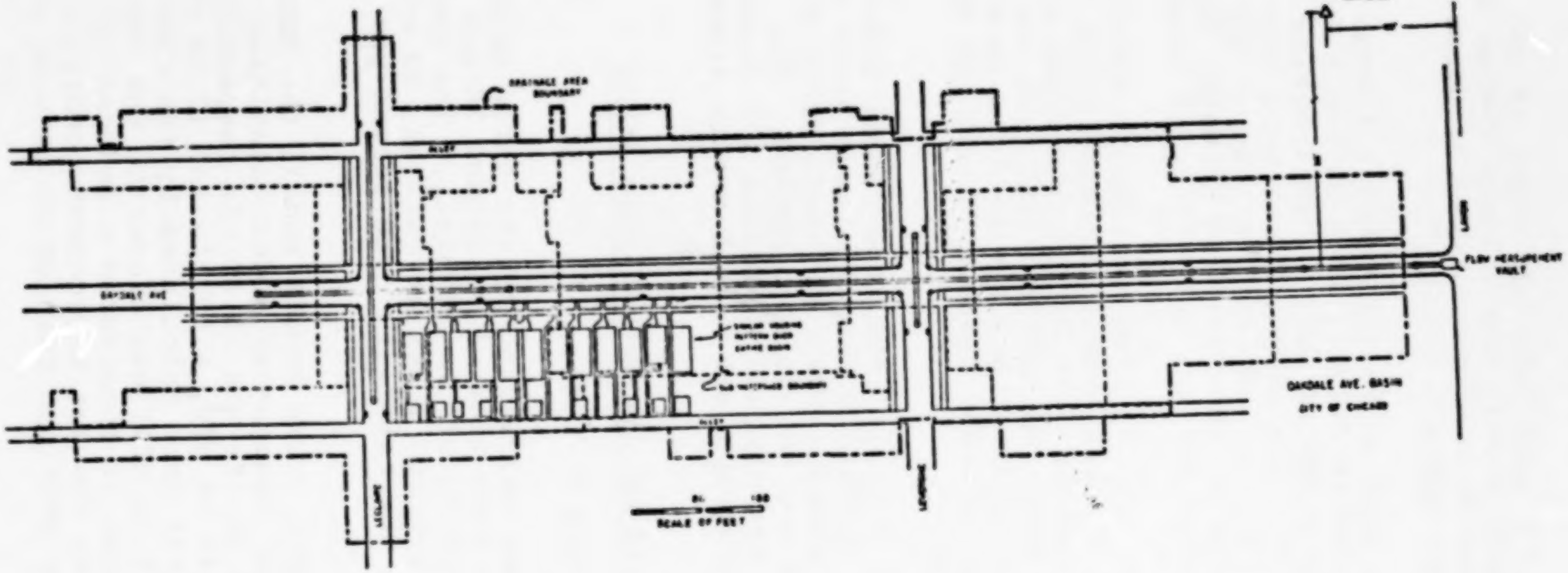

DRAINAGE AREA BOUNDARY
INLET
FLOW MEASUREMENT VAULT
OAKDALE AVE. BASIN
CITY OF CHICAGO
SCALE OF FEET
100

Figure 148. PLAN OF 12.9-ACRE OAKDALE AVENUE DRAINAGE BASIN CHICAGO, ILLINOIS. [32]

basin is presented in Figure 148. In order to use the Cincinnati Urban Runoff Model, the Oakdale Avenue drainage basin has been divided into 6 basic types of subcatchments, 52 gutters and 9 sewer pipes.

Two storms were applied to the Oakdale Avenue drainage basin, the second storm of July 2, 1960 having a duration of 66 minutes and the storm of July 7, 1964 having a duration of 86 minutes. The computations were made at 1-minute intervals. A detailed computer output for this case study is available at the Division of Water Resources, Department of Civil Engineering, University of Cincinnati.

For the storm of July 2, 1960 the runoff hydrograph at the outlet of the Oakdale Avenue drainage basin was calculated by three methods: the Cincinnati Urban Runoff Model, the EPA Storm Water Management Model and the RRL Method. The resulting hydrographs are plotted in Figure 149 together with the recorded hydrograph and the 66-minute storm hyetograph.

For the storm of July 7, 1964, the runoff hydrograph at the outlet of the Oakdale Avenue drainage basin was calculated by three methods: the Cincinnati Urban Runoff Model, the RRL Method and the Chicago Method. The resulted hydrographs are plotted in Figure 150 together with the recorded hydrograph and the 86-minute storm hyetograph.

Comments on the Results of the Second Case Study

(a) Second Storm of July 2, 1960:

1. The two peaks calculated by the three methods occurred at about the same time, i.e. around 38 and 62 minutes from the beginning of the rainfall. The recorded first peak, however occurred sometime earlier, 32 minutes from the beginning of the rainfall. The second peak was not recorded.

2. The Cincinnati Urban Runoff Model, the RRL Method and the EPA Storm Water Management Model calculated a first peak of 18.13 cfs, 14.30 cfs and 15.60 cfs respectively and a second peak of 10.58 cfs, 6.90 cfs and 12.50 cfs respectively. The first peak calculated by the Cincinnati Urban Runoff Model (18.13 cfs) was closer to the recorded peak (17.40 cfs) because it considered a reduced infiltration capacity due to the wet antecedent condition of the soil caused by the previous rain which occured earlier

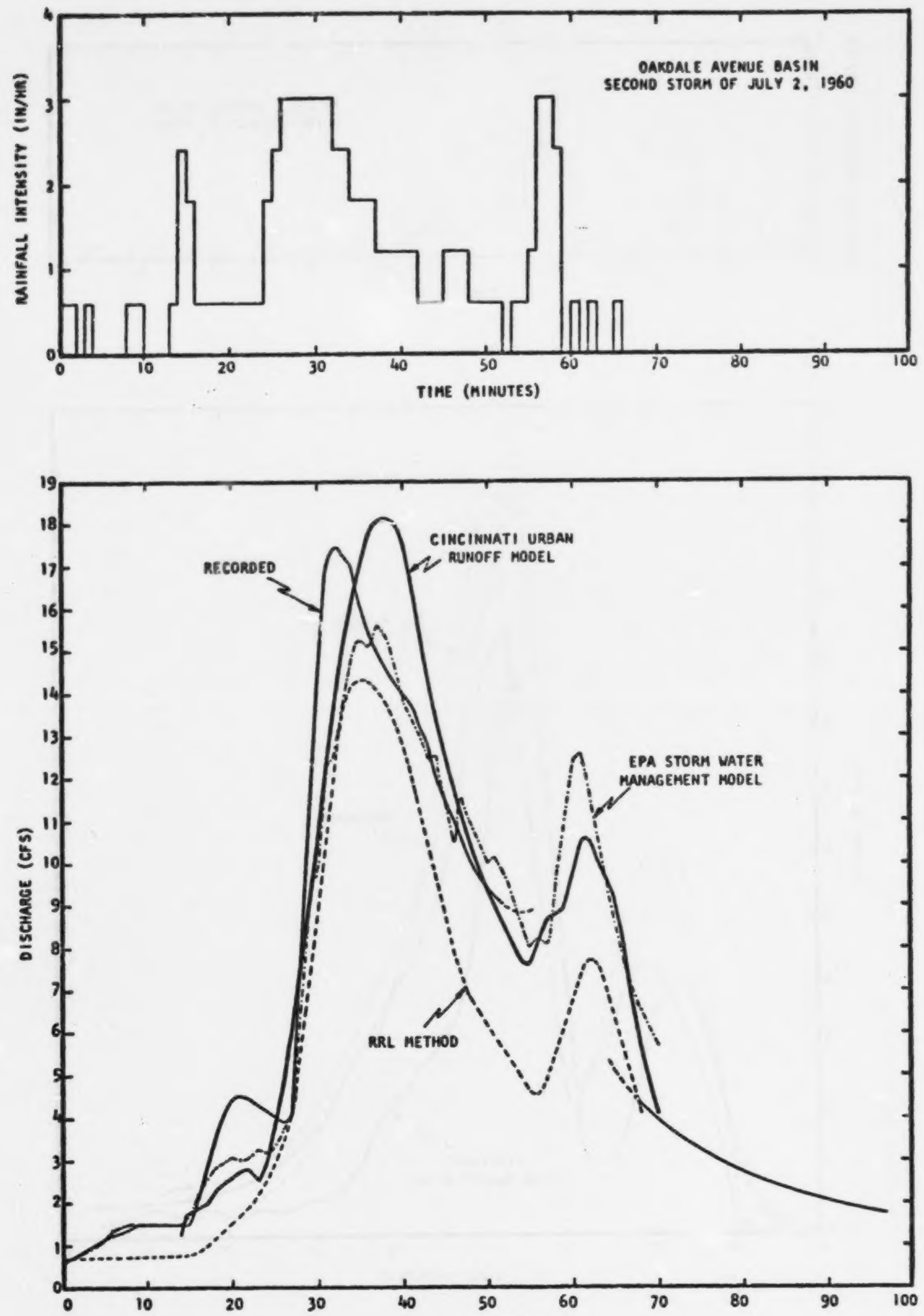


Figure 149. RAINFALL HYETOGRAPH, CALCULATED AND OBSERVED RUNOFF HYDROGRAPHS
OAKDALE AVENUE BASIN, CHICAGO
SECOND STORM OF JULY 2, 1960

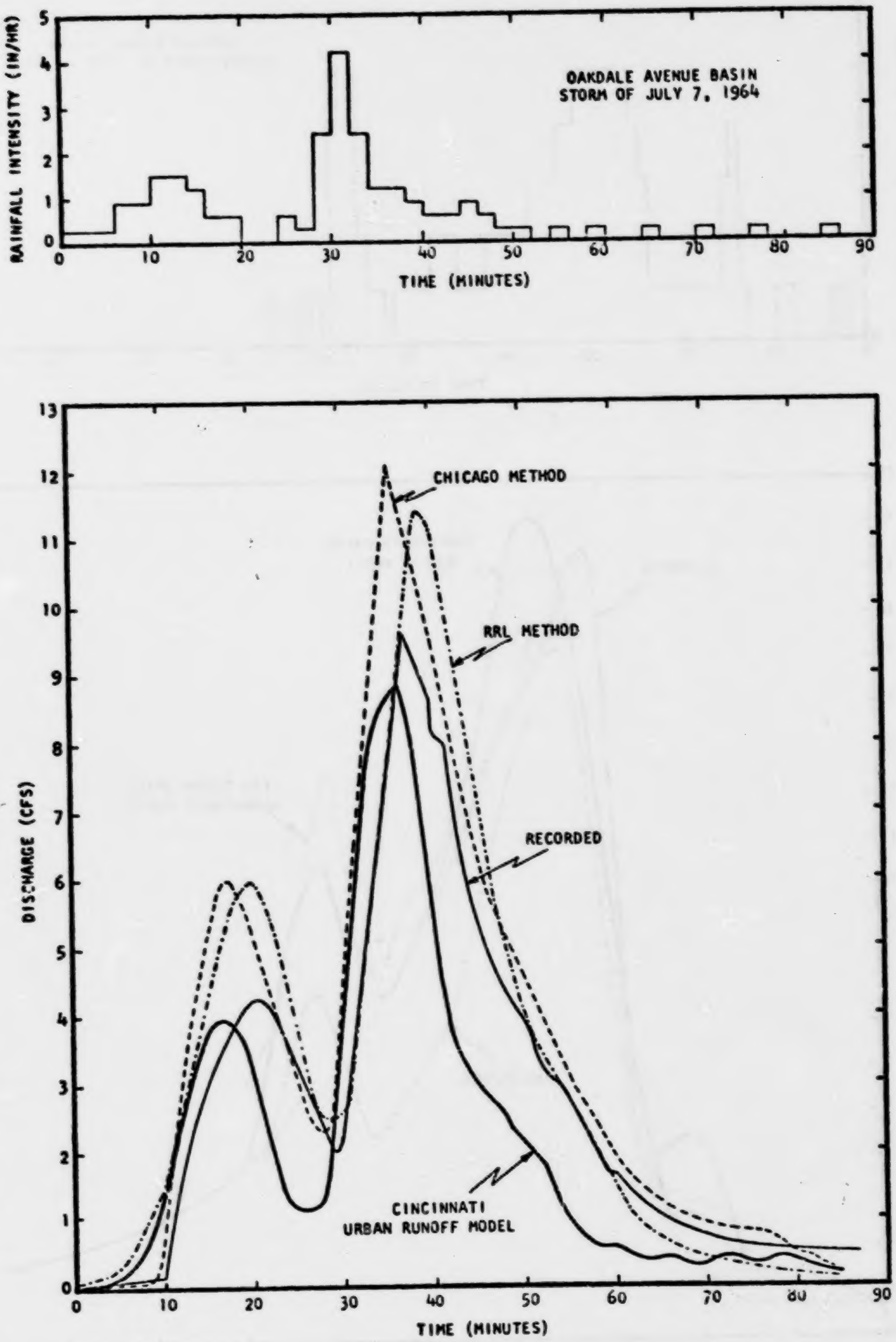


Figure 150. RAINFALL HYETOGRAPH, CLACULATED AND OBSERVED RUNOFF HYDROGRAPHS.
OAKDALE AVENUE BASIN, CHICAGO
STORM OF JULY 7, 1964

the same day. Therefore, some overland flow was generated from the pervious areas and contributed to the first peak. In comparison, the RRL Method without taking into consideration flow from pervious surfaces calculated lower peaks.

3. The three methods gave a comparable first rising limb. However, the RRL Method calculated a lower recession than that obtained by the other two methods, producing a smaller total runoff than that recorded.

(b) Storm of July 7, 1964:

1. The two peaks predicted by the Cincinnati Urban Runoff Model and the Chicago Method occurred at about the same time, i.e. around 17 and 36 minutes after the beginning of the storm. The two peaks predicted by the RRL Method both delayed for about 3 minutes. The recorded peaks occurred 20 and 37 minutes after the beginning of the storm. Therefore the three methods agreed quite satisfactorily in the timing of the peaks.

2. The Cincinnati Urban Runoff Model, the RRL Method and the Chicago Method calculated a first peak of 3.95 cfs, 6.00 cfs and 6.00 cfs respectively, and a second peak of 8.83 cfs, 11.40 cfs and 12.13 cfs respectively. Again the peaks predicted by the Cincinnati Urban Runoff Model (3.95 cfs and 8.83 cfs) were closer to the recorded peaks (4.2 cfs and 9.6 cfs). By using the Cincinnati Urban Runoff Model, the storm under consideration generated a very small amount of overland flow from pervious areas 30 minutes after the beginning of the rainfall and for 10 minutes only. Since the RRL Method does not take into consideration flow from pervious areas and since the overland flow from pervious areas predicted by the Cincinnati Urban Runoff Model was very small, a better agreement between these two methods was expected.

3. The three methods calculated comparable rising limbs for both peaks. However the two recessions calculated by the Cincinnati Urban Runoff Model were considerably lower than the recessions predicted by the other two methods.

11. CONCLUSIONS AND SUGGESTIONS FOR FURTHER STUDY

The Cincinnati Urban Runoff Model consists of five sub-models:

1. The Infiltration Model uses Horton's equation to find the shape of the proper infiltration capacity curve. If data from the urban basin under study are not available, the use of Jens' Infiltration Capacity Curves is recommended. A method has been developed for the computer to shift the infiltration capacity curve in time, so Horton's equation can be used when $i_o < f_o$.

2. The Depression Storage Model uses an equation proposed by Linsley, Kohler, and Paulhus, and a new equation has been developed from it relating rates of depression storage versus time.

3. The Overland Flow Model has been developed using the continuity equation, the momentum equation, and an equation by Crawford relating the depth of flow at the downstream end of the overland flow plane with the surface detention. Several computer runs showed a satisfactory degree of sensitivity of this model to changes of ground slope, ground cover, and length of overland flow.

4. The Gutter Flow Model used the continuity equation for steady flow since the change in depth with respect to time was found negligible. The validity of this simplification is shown in Appendix I.

5. The Routing Model routes the flow through the lateral and main sewer by shifting an inflow hydrograph from one inlet to the next, adds the shifted hydrograph with the new one, and proceeds downstream. This solution does not reduce the peak flows, but it has been proved to compare sufficiently close to the solution obtained from more rigorous methods (i.e., backwater curves computation).

A computer program has been developed and combines the five mentioned sub-models in the sequence with which they appear above.

It is believed that although a great deal of time has been spent in the development of the presented Mathematical Model, each step can and should be improved when more field data are available for verification.

Since depression storage reduces significantly the overland flow, especially from pervious areas, more research is needed to determine the proper depression storage to be used in the Model.

Other factors reducing the overland flow which can be considered in the future are: evaporation, interception, and wetting of surfaces.

More exact storage routing procedures can be developed to route the flow through the gutters and the lateral and main sewers. However, each improvement of the Mathematical Model has to be justified by manpower and computer time needed.

Further research is recommended concerning the capacity of the storm-water gutter inlets, because during heavy rainstorms there exists a carry-over flow (i.e., the portion of the gutter flow that does not go into an inlet).

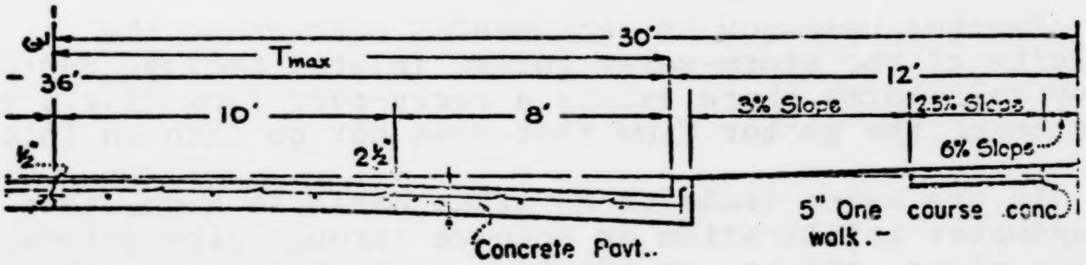
If the water table of an urban basin is high, then groundwater infiltration or seepage through pipe joints, broken pipes, cracks, or openings in manholes and similar faults, should be taken into account. Research can be oriented to find a relation between pipe material, age of the sewer system, and groundwater infiltration or seepage.

The Cincinnati Urban Runoff Model as presented is considered to be preliminary and will be improved as additional testing is accomplished with actual field data.

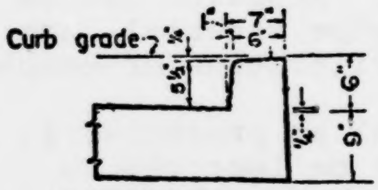
APPENDIX I

DIMENSIONAL ANALYSIS OF GUTTER FLOW

Gutters are usually triangular channels with the one side approximately vertical (this depends on the shape of the curb). As can be seen in Figure 151, a typical battered curb has a height of 5.5 inches and if the water is flowing with this depth then it is covering the corresponding half of the roadway.



30'
T_{max}
36'
12'
10'
8'
3% Slope
2.5% Slope
6% Slope
½"
2½"
5" One course conc. walk.
Concrete Pavt..



Curb grade
1"
7"
6"
5½"
9"
6"
6"

BATTERED CURB

Figure 151.

CONCRETE ROADWAY WITH BATTERED CURB DETAIL

The Rational formula gives:

$$q = c\ i\ \frac{A}{b}$$

where q is the discharge in cfs/ft width

A is the contributing area in acres

b is the width of overland flow (ft)

c is runoff factor < 1

i is average rainfall intensity in inches per hour for a storm duration equal to the time of concentration of the area which provides the overland flow. Usually i is not less than 1 nor more than 3 inches per hour for urban areas.

Through the assumption of orders of magnitude, it is possible to show that the term $[T\frac{dy}{dt}]$ in equation (38) on page 259 can be neglected.

It seems reasonable to assume: [the symbols " = ORD( )" mean "is of the order ( )"]

$$T = \mathrm{ORD}(10^0)\ \mathrm{ft}$$

$$\Delta y = \mathrm{ORD}(10^{-1})\ \mathrm{ft}$$

$$\Delta t = \mathrm{ORD}(10^3)\ \mathrm{sec}$$

$$c = \mathrm{ORD}(10^{-1})$$

$$i = \mathrm{ORD}(10^0)\ \mathrm{in/hr}$$

$$A = \mathrm{ORD}(10^0)\ \mathrm{acres}$$

$$b = \mathrm{ORD}(10^1)\ \mathrm{ft}$$

then the ratio

$$\frac{T\frac{\Delta y}{\Delta t}}{q} = \frac{1}{c}\,\frac{\Delta y/\Delta t}{i}\,\frac{T}{A/b} = \mathrm{ORD}(10^{-2})$$

Therefore the term $[T\frac{\Delta y}{\Delta t}]$ can be neglected from equation (38) as being small in comparison with the rest of the terms of this equation.

As an illustration, for a certain rain, two values of overland flow have been considered in a 5 minute inverval, namely q_1 = 0.00069 cfs/ft and q_2 = 0.00115 cfs/ft at the beginning and the end of the interval. The length of the gutter was 100 ft with a slope 0.03 ft/ft and a Manning's coefficient 0.017. The slope of the street pavement was 1:27.

The corresponding depths of flow have been found by Manning's equation to be y_1 = 0.0774 ft and y_2 = 0.0852 ft. The average surface width of the flow was T = 2.195 ft.

Then $\frac{\Delta y}{\Delta t}\,T = 0.000057\ \frac{\mathrm{cfs}}{\mathrm{ft}}$ which is a 6% of the average inflow q.

APPENDIX II

MANNING'S ROUGHNESS COEFFICIENT

1. For Overland Flow

Manning's n for overland flow can be estimated from the table below [8], or from other more detailed sources [22].

Watershed Cover	Manning's n For Overland Flow
Smooth asphalt	0.012
Asphalt or concrete paving	0.014
Packed clay	0.030
Light turf	0.200
Dense turf	0.350
Dense shrubbery & forest litter	0.400

When there is a mixture of ground cover in a subcatchment of area A, then a harmonic mean Manning's roughness coefficient n can be obtained from the expression:

$$\frac{A}{n} = \frac{B}{n_1} + \frac{(A - B)}{n_2}$$

where B is a part of the subcatchment area having a Manning's roughness coefficient n_1 and the rest of the subcatchment area (A - B) has a roughness coefficient equal to n_2 [8].

The same procedure can be used if an area has more than two different kinds of ground cover.

2. For Pipe Flow

For a combined sewer system, the same coating of slimes form on the inside of all materials used for the construction of the sewers. This means that all materials used in the construction of combined sewers will have the same hydraulic roughness coefficient [31]. In Manning's formula that coefficient is n = 0.013. Proof of this was demonstrated by extensive tests at the Ohio State University [33].

APPENDIX III

COMPUTER PROGRAM FOR THE CINCINNATI URBAN RUNOFF MODEL

The volume of computations required by the Cincinnati Urban Runoff Model would have been too formidable without the use of an electronic computer. A digital computer IBM 1130 with an available storage of 16K was used to run parts of the present program. Because usually more storage is required, the whole program was run on the digital computer IBM 360/65, of the University of Cincinnati, which has a maximum storage capacity of 512K. The computer program is written in FORTRAN IV language.

The differences in programming for the IBM 1130 and the IBM 360/65 are only:

	IBM 1130	IBM 360/65
1.	job cards	job cards
2.	READ (2, ...)	READ (5, ...)
3.	WRITE (3, ...)	WRITE (6, ...)
4.	Only arithmetic IF statements	Both logical and arithmetic IF statements

The program presented here consists of three parts, the main program and two subroutines, namely:

1/ MAIN program: for infiltration, depression storage, and overland flow

2/ GUTFL subroutine: for the gutter flow

3/ PIROU subroutine: for the routing through the lateral and main sewers

"CINCINNATI URBAN RUNOFF MODEL"

COMPUTER PROGRAM

```
C
C     *****************************************************
C     INFILTRATION AND DEPRESSION STORAGE IN PERVICUS AREAS
C     *****************************************************
C
C
C     DIVIDE THE TOTAL RAINFALL TIME IN N EQUAL SMALL INCREMENTS
C     READ N+1 VALUES OF AVAILABLE RAINFALL HYDROGRAPH(IN/HR)
C     EACH TIME INCREMENT DT IS IN MINUTES
C     AT TIME ZERO I=0. AT THE END OF THE TIME PERIOD I=N
C     RAINFALL INTENSITY AT TIME ZERO IS RO(FOR I=0)
C     RAINFALL INTENSITY AT A TIME T=DT*I IS RI(I)
C     AT A TIME T=DT*I,INFILTRATION CAPACITY IS F(I) AND ACTUAL
C     INFILTRATION IS AF(I) IN (IN/HR)
C     THE CONSTANT RATE AT WHICH F(I) APPROACHES ASYMPTOTICALLY
C     AS TIME CONTINUES,IS FC
C     THE RATE OF INFILTRATION CAPACITY WHEN TIME=0(I.E. I=0) IS FO
C     CK IS A POSSITIVE CONSTANT
C     DIFFERENCE BETWEEN RAINFALL AND ACTUAL INFILTRATION  S(I)
C     ACCUMULATED MASS PRECIPITATED UNTIL A TIME DT*I IS RM(I) INCHES
C     ACCUM.MASS ACTUALLY INFILTRATED UNTIL A TIME DT*I IS AFM(I) INCHES
C     WHERE RM(I) OR AFM(I) INDICATE THE MASS FROM TIME DT*(I-1) UNTIL
C     TIME DT*I ,PLUS THE PREVIOUS MASS
C     DIFFERENCE BETWEEN MASSES UNTIL A TIME DT*I IS P(I) IN INCHES
C     VOLUME OF WATER IN DEPRESSION STORAGE IS VS(I) INCHES
C     TOTAL DEPRESSION STORAGE CAPACITY SD INCHES
C     DEPRESSION STORAGE SUPPLY IS D(I)  INCH/HOUR
C     SS(I) IS THE RATE OF (RAIN-INFILTR-DEPRESSION STORAGE)
C     PP(I) IS THE ACCUMULATED MASS OF(RAIN-INFILTR-DEPRESSION STORAGE)
      COMMON    RI(70),AF(70),S(70),RM(70),AFM(70),P(70),D(70),VS(70),SS
     1(70),PP(70),SSIM(70),PPIM(70),VSIM(70),DIM(70),DATE(8),DC(70),DE(7
     20),OVQ(250,30),OVQW(70),NEW(30),QINL(70,20),QINLO(20),IW(250),V(25
     30),QLEAV(250),N,DT,NN,NINL,QINP(250,20)
      WRITE(6,1)
    1 FORMAT('1')
      READ(5,21)(DATE(I),I=1,8),DT,N
   21 FORMAT(8A4,F5.2,I5)
      READ (5,4) RO
    4 FORMAT(F5.2)
      READ(5,5) (RI(I),I=1,N)
    5 FORMAT (12F5.2)
      READ (5,6) FC,FO,CK,SD,SDIM
    6 FORMAT(3F10.5,2F8.4)
      READ(5,20)AN
   20 FORMAT(F5.2)
      WRITE(6,501)
  501 FORMAT(5X,'*******************************************************
     1******',/,5X,'SIMULATION OF INFILTR.AND DEPRESSION STORAGE IN PERV
     2IOUS AREA',/,5X,'*************************************************
     3************',//)
      WRITE(6,22)(DATE(I),I=1,5)
   22 FORMAT(5X,'RAIN OF',6X,5A4,//)
      WRITE(6,23)(DATE(I),I=6,8)
   23 FORMAT(5X,'TIME OF RAIN START',3X,3A4,//)
```

```
      WRITE (6,605)
  605 FORMAT(11X,'FC',8X,'FO',9X,'CK')
      WRITE(6,606)FC,FO,CK
  606 FORMAT(10X,F4.2,6X,F4.2,6X,F6.4,//)
      WRITE(6,607) SD
  607 FORMAT(5X,'TOTAL DEPRESSION STORAGE CAPACITY OF THE',/,5X,'SUBCATC
     1HMENT IS   (IN INCHES)  =',15X,F8.4,//)
      IF (RO-FO) 10,7,7
    7 DO 8 I=1,N
    8 AF(I)=FC+((FO-FC)*(2.718**(-CK*DT*I)))
      SO=RO-FO
      DO 9 I=1,N
    9 S(I)=RI(I)-AF(I)
   10 RM(1)=((RO+RI(1))/2.0)*DT/60.0
      IF (RO-FO) 19,12,12
   12 DO 11 I=2,N
   11 RM(I)=RM(I-1)+((RI(I)+RI(I-1))/2.0)*DT/60.0
      DO 13 I=1,N
   13 AFM(I)=((FC/60.0)*DT*I)+(((FO-FC)/(60.0*CK))*(1.0-(2.718**(-CK*DT*
     1I))))
      DO 14 I=1,N
   14 P(I)=RM(I)-AFM(I)
      OD=SO
      SSO=0.000
      NCON=0.0
      DO 800 I=1,N
      IF(RI(I)-AF(I)) 900,900,799
  900 D(I)=0.000
      NCON=1.0
      VS(I)=SD
      SS(I)=S(I)
      PP(I)=P(I)-SD
      GO TO 800
  799 IF(NCON) 798,798,900
  798 D(I)=S(I)*(2.718**(-P(I)/SD))
      VS(I)=SD*(1.0-(2.718**(-P(I)/SD)))
      SS(I)=S(I)-D(I)
      PP(I)=P(I)-VS(I)
  800 CONTINUE
      WRITE (6,15)
   15 FORMAT(5X,'TIME FROM    RAINFALL        ACTUAL       DEPRESSION
     1OVERLAND ',/,5X,'RAIN START   INTENSITY   INFILTRATION   STOR.SUPP
     2LY   FLOWSUPPLY',/,5X,'(MINUTES)       (IN/HR)        (IN/HR)       (
     3IN/HR)       (IN/HR)  ',/)
      T=0.00
      WRITE(6,16) T,RO,FO,OD,SSO
   16 FORMAT(5X,F8.2,6X,F6.3,8X,F6.3,8X,F6.3,8X,F6.3)
      DO 17 I=1,N
      TT=DT*I
   17 WRITE(6,18) TT,RI(I),AF(I),D(I),SS(I)
   18 FORMAT(5X,F8.2,6X,F6.3,8X,F6.3,8X,F6.3,8X,F6.3)
      WRITE(6,805)
  805 FORMAT('1')
      WRITE(6,806)
  806 FORMAT(5X,'TIME FROM          MASS             MASS       DEPRESSION
     1DIFFERENCE',/,5X,'RAIN START   PRECIPITATED   INFILTRATED     STORA
     2GE      (MR-MI-DS)',/,5X,'(MINUTES)          (IN)          (IN)
     3     (IN)           (IN)     ',/)
      DO 807 I=1,N
      TT=DT*I
  807 WRITE(6,808) TT,RM(I),AFM(I),VS(I),PP(I,
  808 FORMAT(5X,F8.2,8X,F6.3,9X,F6.3,7X,F6.4,7X,F6.3,
      GO TO 118
C     IF INFILTRATION CAPACITY GREATER THAN RAINFALL INTENSITY
C     AT THE BEGINNING OF THE RAINFALL (WHERE TIME=0 AND I=0) THEN
C     INFILTRATION CAPACITY CURVE STARTS AFTER A TIME (I*DT+X*L)FROM
C     THE BEGINNING OF THE RAINFALL
C     WE HAVE DIVIDED DT IN (NN+1) EQUAL INCREMENTS X(L FROM 1 TO NN)
C     THE SHIFTED INFILTRATION CAPACITY CURVE WILL INTERSECT THE
C     RAINFALL INTENSITY CURVE AFTER A TIME TJ=J*DT+Y AFTER THE
C     BEGINNING OF THE INFILTRATION CAPACITY CURVE,OR AFTER A TIME
C     (I*DT+X*L) FROM THE BEGINNING OF THE RAINFALL
C     THE I AFTER WHICH THE SECTION OF THE CURVES TAKES PLACE IS(ISEC)
   19 X=DT/AN
      NN=AN-1
      I=1
      DO 299 L=1,NN
      ALLL=((RI(I)+(X*L*(RI(I+1)-RI(I))/DT)-FC)/(FO-FC))
      IF(ALLL.GT.0.AND.ALLL.LT.1.) GO TO 9999
      GO TO 299
 9999 TJ=-(1.0/CK)*ALOG((RI(I)+(X*L*(RI(I+1)-RI(I))/DT)-FC)/(FO-FC))
      B1=RM(I)+(RI(I)+(X*L*(RI(I+1)-RI(I))/(2.0*DT)))*(X*L/60.0)
      B2=(FC*TJ/60.0)+((FO-FC)*(1.0-(2.718**(-CK*TJ)))/(60.0*CK))
      ISEC=I
      LSEC=L
      B12T=ABS(B1-B2)
      IF(B12T-0.01) 301,301,299
  299 CONTINUE
      DO 300 I=2,N
      DO 300 L=1,NN
      RM(I)=RM(I-1)+((RI(I)+RI(I-1))/2.0)*DT/60.0
      ALLL=((RI(I)+(X*L*(RI(I+1)-RI(I))/DT)-FC)/(FO-FC))
      IF(ALLL.GT.0.AND.ALLL.LT.1.) GO TO 9998
      GO TO 300
 9998 TJ=-(1.0/CK)*ALOG((RI(I)+(X*L*(RI(I+1)-RI(I))/DT)-FC)/(FO-FC))
      B1=RM(I)+(RI(I)+(X*L*(RI(I+1)-RI(I))/(2.0*DT)))*(X*L/60.0)
      B2=(FC*TJ/60.0)+((FO-FC)*(1.0-(2.718**(-CK*TJ)))/(60.0*CK))
      ISEC=I
      LSEC=L
      B12T=ABS(B1-B2)
      IF(B12T-0.01) 301,301,300
  300 CONTINUE
C     BEFORE THE SECTION
  301 AFO=RO
      SO=0.0
      OD=0.000
      SSO=0.000
      DO 100 I=1,ISEC
      AF(I)=RI(I)
      S(I)=0.0
      D(I)=0.000
  100 SS(I)=0.000
```

```
C     AT THE SECTION
      IISEC=ISEC+1
      RISEC=(LSEC*X*(RI(IISEC)-RI(ISEC))/DT)+RI(ISEC)
      AFSEC=RISEC
      SSEC=0.0
      DSEC=0.000
      SSSEC=0.000
C     AFTER THE SECTION
      IISEC=ISEC+1
      DO 101 I=IISEC,N
      AF(I)=FC+((FO-FC)*(2.718**(-CK*(TJ+DT*(I-ISEC)-X*LSEC))))
  101 S(I)=RI(I)-AF(I)
C     BEFORE THE SECTION
      DO 102 I=2,ISEC
  102 RM(I)=RM(I-1)+((RI(I)+RI(I-1))/2.0)*DT/60.0
      DO 103 I=1,ISEC
      AFM(I)=RM(I)
      VS(I)=0.0000
      PP(I)=0.000
  103 P(I)=AFM(I)-RM(I)
C     AT THE SECTION
      RMSEC=RM(ISEC)+((RI(ISEC)+RISEC)*X*LSEC)/(2.0*60.0)
      AFMSE=RMSEC
      PSEC=0.0
      VSSEC=0.0000
      PPSEC=0.000
C     AFTER THE SECTION
      DO 104 I=IISEC,N
      ISEC=IISEC-1
  104 RM(I)=RM(I-1)+((RI(I)+RI(I-1))/2.0)*DT/60.0
      DO 105 I=IISEC,N
  105 AFM(I)=(FC/60.0)*((I-ISEC)*DT-X*LSEC+TJ)+((FO-FC)/(60.0*CK))*(1.0-
     1(2.718**(-CK*((I-ISEC)*DT-X*LSEC+TJ))))
      DO 205 I=IISEC,N
  205 P(I)=RM(I)-AFM(I)
      NCON=0.0
      DO 810 I=IISEC,N
      IF(RI(I)-AF(I)) 901,901,795
  901 D(I)=0.000
      NCON=1.0
      VS(I)=SD
      SS(I)=S(I)
      PP(I)=P(I)-SD
      GO TO 810
  795 IF(NCON) 794,794,901
  794 D(I)=S(I)*(2.718**(-P(I)/SD))
      VS(I)=SD*(1.0-(2.718**(-P(I)/SD)))
      SS(I)=S(I)-D(I)
      PP(I)=P(I)-VS(I)
  810 CONTINUE
      T=DT*ISEC+X*LSEC-TJ
      WRITE(6,106) T
  106 FORMAT(5X,'INFILTRATION CAPACITY CURVE STARTS AFTER THE',/,5X,'BEG
     1INNING OF THE  RAINFALL (IN MINUTES)  =',5X,F8.2,//)
      WTT=DT*ISEC+X*LSEC
      WRITE(6,108)WTT
  108 FORMAT(5X,'INFILTRATION CAPACITY CURVE INTERSECTS  THE',/,5X,'RAIN
     1FALL INTENSITY CURVE AFTER THE BEGINNING',/,5X,'OF THE RAINFALL
     2  (IN MINUTES)  =',12X,F8.2,//)
      WRITE (6,110)
  110 FORMAT(5X,'TIME FROM    RAINFALL         ACTUAL      DEPRESSION
     1OVERLAND ',/,5X,'RAIN START   INTENSITY   INFILTRATION   STOR.SUPP
     2LY   FLOWSUPPLY',/,5X,'(MINUTES)      (IN/HR)       (IN/HR)      (
     3IN/HR)       (IN/HR)  ',/)
      W=0.00
      WRITE(6,111)W,RO,AFO,OD,SSO
  111 FORMAT(5X,F8.2,6X,F6.3,8X,F6.3,8X,F6.3,8X,F6.3)
      DO 112 I=1,ISEC
      WW=DT*I
  112 WRITE(6,113)WW,RI(I),AF(I),D(I),SS(I)
  113 FORMAT(5X,F8.2,6X,F6.3,8X,F6.3,8X,F6.3,8X,F6.3)
      WRITE(6,114)WTT,RISEC,AFSEC,DSEC,SSSEC
  114 FORMAT(5X,F8.2,6X,F6.3,8X,F6.3,8X,F6.3,8X,F6.3)
      DO 115 I=IISEC,N
      WWW=DT*I
  115 WRITE(6,116) WWW,RI(I),AF(I),D(I),SS(I)
  116 FORMAT(5X,F8.2,6X,F6.3,8X,F6.3,8X,F6.3,8X,F6.3)
      WRITE(6,120)
  120 FORMAT('1')
      WRITE(6,121)
  121 FORMAT(5X,'TIME FROM          MASS               MASS       DEPRESSION
     1DIFFERENCE',/,5X,'RAIN START   PRECIPITATED   INFILTRATED    STORA
     2GE      (MR-MI-DS)',/,5X,'(MINUTES)          (IN)           (IN)
     3    (IN)           (IN)    ',/)
      DO 122 I=1,ISEC
      WW=DT*I
  122 WRITE(6,123) WW,RM(I),AFM(I),VS(I),PP(I)
  123 FORMAT(5X,F8.2,8X,F6.3,9X,F6.3,7X,F6.4,7X,F6.3)
      WRITE(6,124)WTT,RMSEC,AFMSE,VSSEC,PPSEC
  124 FORMAT(5X,F8.2,8X,F6.3,9X,F6.3,7X,F6.4,7X,F6.3)
      DO 125 I=IISEC,N
      WWW=DT*I
  125 WRITE(6,126) WWW,RM(I),AFM(I),VS(I),PP(I)
  126 FORMAT(5X,F8.2,8X,F6.3,9X,F6.3,7X,F6.4,7X,F6.3)
  118 WRITE (6,119)
  119 FORMAT ('1')
C
C     ********************************************************
C     INFILTRATION AND DEPRESSION STORAGE IN IMPERVIOUS AREAS
C     ********************************************************
C
C     INFILTRATION CAPACITY IS ZERO
C     VOLUME OF WATER IN DEPRESSION STORAGE IS VSIM(I) INCHES
C     TOTAL DEPRESSION STORAGE CAPACITY IS SDIM (INCH)
C     DEPRESSION STORAGE SUPPLY IS DIM(I) INCH/HOUR
C     SSIM(I) IS THE RATE OF (RAINFALL-DEPR.STORAGE)
C     PPIM(I) IS THE ACCUMULATED MASS OF (RAIN-DEPR.STORAGE)
      WRITE(6,500)
  500 FORMAT(5X,'*******************************************************
     1********',/,5X,'SIMULATION OF INFILTR.AND DEPRESSION STORAGE IN IM
     2PERVIOUS AREA',/,5X,'*********************************************
     3*******************',//)
```

```
      WRITE(6,25)
   25 FORMAT(5X,'INFILTRATION CAPACITY IS ZERO',//)
      WRITE(6,26)SDIM
   26 FORMAT(5X,'TOTAL DEPRESSION STORAGE CAPACITY OF THE',/,5X,'SUBCATC
     1HMENT IS   (IN INCHES)  =',15X,F8.4,//)
      DO 28 I=1,N
      DIM(I)=RI(I)*(2.718**(-RM(I)/SDIM))
      VSIM(I)=SDIM*(1.0-(2.718**(-RM(I)/SDIM)))
      SSIM(I)=RI(I)-DIM(I)
      PPIM(I)=RM(I)-VSIM(I)
   28 CONTINUE
      WRITE(6,29)
   29 FORMAT(5X,'TIME FROM    RAINFALL    DEPRESSION    OVERL.FLOW',/,5X
     1,'RAIN START   INTENSITY   STOR.SUPPLY     SUPPLY',/,5X,'(MINUTES)
     2    (IN/HR)      (IN/HR)      (IN/HR)',/)
      TIMO=0.00
      DIMO=RO
      SSIMO=0.000
      WRITE(6,37)TIMO,RO,DIMO,SSIMO
   37 FORMAT(5X,F8.2,6X,F6.3,7X,F6.3,8X,F6.3)
      DO 30 I=1,N
      TIM=DT*I
   30 WRITE(6,31) TIM,RI(I),DIM(I),SSIM(I)
   31 FORMAT(5X,F8.2,6X,F6.3,7X,F6.3,8X,F6.3)
      WRITE(6,32)
   32 FORMAT('1')
      WRITE(6,33)
   33 FORMAT(5X,'TIME FROM        MASS       DEPRESSION   DIFFERENCE',/,
     15X,'RAIN START   PRECIPITATED    STORAGE      (RM-DS)',/,5X,'(MINU
     2TES)        (IN)          (IN)        (IN)',/)
      DO 34 I=1,N
      TTIM=DT*I
   34 WRITE(6,35) TTIM,RM(I),VSIM(I),PPIM(I)
   35 FORMAT(5X,F8.2,8X,F6.3,7X,F6.4,7X,F6.3)
      WRITE(6,36)
   36 FORMAT('1')
C
C     ***************************
C     SIMULATION OF OVERLAND FLOW
C     ***************************
C
C     USE AS DATA THE OVERLAND FLOW SUPPLY FROM THE
C     INFILTRATION AND DEPRESSION STORAGE PROGRAM
C     ARW IS THE MANNING'S ROUGHNESS COEFFICIENT OF THE GROUND
C     ALW IS THE LENGTH OF THE OVERLAND FLOW IN FEET
C     SLW IS THE SLOPE OF THE SUBCATCHMENT IN THE DIRECTION OF FLOW
C     NW IS THE NUMBER OF THE SUBCATCHMENTS
C     INDW=1 INDICATES PERVIOUS SUBCATCHMENT,INDW=2 IMPERVIOUS
C     DE=DETENTION STORAGE AT EQUILIBRIUM (INCHES)
C     DC IS THE CURRENT DETENTION STORAGE(INCHES)
C     OVQ(I,IW) IS THE OVERLAND FLOW RATE OF THE SUBCATCHMENT OF
C     NUMBER IA,AT A TIME I*DT,IN (CFS/FT OF WIDTH)
C     OVQW(I) IS THE SAME OVERLAND FLOW IN(IN/HR/UNIT AREA)
      READ(5,650) NW
  650 FORMAT(15)
      DO 651 IA=1,NW
      READ(5,649)IA,INDW,ALW,ARW,SW
  649 FORMAT(2I2,3F7.4)
      WRITE(6,652)
  652 FORMAT(/,5X,'*****************************************************
     1**')
      IF(INDW-1) 653,654,653
  653 WRITE(6,655)IA
  655 FORMAT(5X,'OVERLAND FLOW SIMULATION OF IMPERVIOUS AREA NUMBER ',I2
     1)
      GO TO 657
  654 WRITE(6,656)IA
  656 FORMAT(5X,'OVERLAND FLOW SIMULATION OF  PERVIOUS AREA NUMBER ',I2)
  657 WRITE(6,658)
  658 FORMAT(5X,'*******************************************************
     1',//)
      WRITE(6,659)ALW
  659 FORMAT(5X,'LENGTH OF OVERLAND FLOW IN FEET         =',F7.2)
      WRITE(6,690) SW
  690 FORMAT(5X,'SLOPE AT THE DIRECTION OF FLOW (FT/FT) =',F7.4)
      WRITE(6,691) ARW
  691 FORMAT(5X,'MANNINGS  ROUGHNESS  COEFFICIENT        =',F7.4,//)
      AAA=510.35*(SW**0.5)/(ARW*ALW)
      ABB=60.0/DT
      AB=1.6**1.6666
      BA=AAA**0.6
      WRITE(6,692)
  692 FORMAT(5X,'TIME FROM       OVERLAND FLOW AT       OVERLAND FLOW AT',
     1/,5X,'RAIN START      THE DOWNSTREAM         THE DOWNSTREAM',/,5X,'
     2  IN            END OF WATERSHED       END OF WATERSHED',/,5X,' MINUT
     3ES       (IN/HR/UNIT AREA)      (CFS/FT OF WIDTH)',/)
      IV=IISEC
      OVV=WTT
      IF(INDW-1) 680,681,680
  680 WTT=0.00
      IISEC=1
      DO 682 I=1,N
  682 SS(I)=SSIM(I)
      GO TO 683
  681 IF(RO-FO) 683,684,684
  684 WTT=0.00
      IISEC=1
  683 DESEC=0.0
      OVQO=0.00000
      OVQWO=0.000
      WRITE(6,850)WTT,OVQHO,OVQO
  850 FORMAT(7X,F6.2,11X,F7.4,14X,F8.6)
      IF(IISEC-1) 678,677,678
  678 DO 849 I=1,IISEC
  849 OVQ(I,IA)=0.00000
  677 DO 660 I=IISEC,N
      IF(SS(I)-0.0) 6659,6659,6600
 6659 DE(I)=0.0
      GO TO 660
 6600 DE(I)=0.009792*(((SS(I)*ARW*ALW)**6.0)**0.1)/(SW**0.3)
  660 CONTINUE
      I=IISEC
      IF(SS(I)-0.02)830,830,840
  830 DC(I)=0.0
      OVQW(I)=0.000
      OVQ(I,IA)=0.00000
      GO TO 829
  840 STA=0.5*SS(I)
      AA=0.6/(DE(I)**3.0)
      ABA=AA*BA
      DTO=(DT*I)-WTT
      BBA=60.0/DTO
      IF(INDW-1) 5010,5020,5010
 5010 DO 851 ID=1,20000,1
      WD=ID
      G=WD/10000.0
      GO TO 5000
 5020 DO 851 ID=1,2000,1
      WD=ID
      G=WD/1000.0
 5000 STB=(((BA*G)+(ABA*(G**4.0)))**1.6666)+(BBA*G)
      IF(STB-STA)851,852,852
  851 CONTINUE
  852 DC(I)=G
      OVQW(I)=(2.0*AAA)*(DC(I)**1.6666)*((1.0+(AA*(DC(I)**3.0)))**1.6666
     1)
      OVQ(I,IA)=ALW*OVQW(I)/43200.0
  829 TIME=DT*I
      WRITE(6,853) TIME,OVQW(I),OVQ(I,IA),DE(I),DC(I),SS(I)
  853 FORMAT(7X,F6.2,11X,F7.4,14X,F8.6,10X,F8.5,10X,F8.5,10X,F6.3)
      NWN=N+8
      NSTAR=N+1
      DO 661 I=NSTAR,NWN
      SS(I)=SS(N)
  661 DE(I)=DE(N)
      KKW=NWN-1
      SUM=0.0
      DO 663 I=IISEC,KKW
      STA=(0.5*(SS(I)+SS(I+1)-OVQW(I)))+(ABB*DC(I))
      K=I+1
      IF(STA-0.0) 1652,1652,9888
 9888 IF(DE(K)-0.001) 666,662,662
  662 AA=0.6/(DE(K)**3.0)
      ABA=AA*BA
      IF(INDW-1) 5030,1650,5030
 5030 IF(SW-0.5) 5029,5028,5028
 5029 DO 664 ID=1,20000,1
      WD=ID
      G=WD/10000.0
      STB=(((BA*G)+(ABA*(G**4.0)))**1.6666)+(ABB*G)
      IF(STB-STA) 664,665,665
  664 CONTINUE
 5028 DO 679 ID=1,20000,1
      WD=ID
      G=WD/100000.0
      STB=(((BA*G)+(ABA*(G**4.0)))**1.6666)+(ABB*G)
      IF(STB-STA) 679,665,665
  679 CONTINUE
```

```
1650 DO 646 ID=1,2000,1
     WD=ID
     G=WD/1000.0
     STB=(((BA*G)+(ABA*(G**4.0)))**1.6666)+(ABB*G)
     IF(STB-STA) 646,665,665
 646 CONTINUE
 665 DC(K)=G
     IF(DE(K)-DC(K)) 666,667,667
 666 IF(INDW-1) 5060,1651,5060
5060 IF(SW-0.5) 5027,5026,5026
5027 DO 668 ID=1,20000,1
     WD=ID
     G=WD/10000.0
     STB=(AAA*(G**1.6666)*AB)+(ABB*G)
     IF(STB-STA) 668,669,669
 668 CONTINUE
5026 DO 5025 ID=1,20000,1
     WD=ID
     G=WD/100000.0
     STB=(AAA*(G**1.6666)*AB)+(ABB*G)
     IF(STB-STA) 5025,669,669
5025 CONTINUE
1651 DO 644 ID=1,2000,1
     WD=ID
     G=WD/1000.0
     STB=(AAA*(G**1.6666)*AB)+(ABB*G)
     IF(STB-STA) 644,669,669
 644 CONTINUE
 669 DC(K)=G
     GO TO 676
1652 DC(K)=0.0
     OVQW(K)=0.000
     GO TO 670
 676 OVQW(K)=(2.0*AAA)*(DC(K)**1.6666)*AB
     GO TO 670
 667 OVQW(K)=(2.0*AAA)*(DC(K)**1.6666)*((1.0+(AA*(DC(K)**3.0)))**1.6666
    1)
 670 OVQ(K,IA)=ALW*OVQW(K)/43200.0
     TIME=K*DT
     WRITE(6,671) TIME,OVQW(K),OVQ(K,IA),DE(K),DC(K),SS(K)
 671 FORMAT(7X,F6.2,11X,F7.4,14X,F8.6,10X,F8.5,10X,F8.5,10X,F6.3)
     SUM=SUM+((OVQW(I)+OVQW(K))*DT/120.0)
     IF(K-N) 663,672,672
 672 IF(OVQW(K)-0.0009) 673,663,663
 663 CONTINUE
 673 NEW(IA)=K
     NNNN=NEW(IA)+1
     DO 6556 I=NNNN,NWN
     OVQ(I,IA)=0.00000
6556 CONTINUE
     WRITE(6,674) SUM
 674 FORMAT(//,5X,'TOTAL VOLUME(INCHES)WHICH RAN-OFF IS =',F7.3)
     IISEC=IV
     WTT=OVV
     WRITE(6,675)
 675 FORMAT('1')
 631 CONTINUE
     CALL GUTFL
     CALL PIROU
     CALL EXIT
     END
```

```
      SUBROUTINE GUTFL
C
C     **********************
C     GUTTER  FLOW  PROGRAMM
C     **********************
C
C     NINL IS THE NUMBER OF INLETS
C     N IS THE NUMBER OF TIME INCREMENTS
C     NW IS THE NUMBER OF SUBCATCHMENTS
C     THE PROGRAM WILL TREAT INLETS ACCEPTING A NUMBER OF
C     NBRAN  SYSTEMS OF GUTTERS,AND DIRECT CONNECTIONS
C     N1 IS THE NUMBER OF THE FIRST UPSTREAM GUTTER OF
C     EACH GUTTER SYSTEM
C     N2 IS THE NUMBER OF THE LAST DOWNSTREAM GUTTER OF
C     EACH GUTTER SYSTEM
C     ITYPL IS THE NUMBER OF SUBCATCHMENT FEEDING  A  GUTTER
C     FROM THE LEFT SIDE
C     ITYPR IS THE NUMBER OF THE SUBCATCHMENT FEEDING  A  GUTTER
C     FROM THE RIGHT SIDE
C     GLEN IS THE LENGTH OF THE GUTTER
C     IROOF IS THE NUMBER OF HOUSE ROOFS DIRECTLY CONNECTED WITH AN INLET
C     IWRF IS THE NUMBER OF THE TYPICAL HOUSE ROOF SUBCATCHMENT
C     ROOFL IS THE WIDTH OF A ROOF IN FEET
      COMMON    RI(70),AF(70),S(70),RM(70),AFM(70),P(70),D(70),VS(70),SS
     1(70),PP(70),SSIM(70),PPIM(70),VSIM(70),DIM(70),DATE(8),DC(70),DE(7
     20),OVQ(250,30),OVQW(70),NEW(30),QINL(70,20),QINLO(20),IW(250),V(25
     30),QLEAV(250),N,DT,NN,NINL,QINP(250,20)
      N=N+8
      READ(5,3000) NINL
 3000 FORMAT(I5)
      DO 3004 IIN=1,NINL
      DO 3004 I=1,N
 3004 QINL(I,IIN)=0.0
      DO 4001 IIN=1,NINL
      READ(5,3009) NBRAN
 3009 FORMAT(I5)
      DO 4010 I1=1,NBRAN
      READ(5,3010) N1,N2
 3010 FORMAT(2I5)
      DO 4002 J=N1,N2
      READ(5,3011) ITYPL,ITYPR,GLEN
 3011 FORMAT(2I5,F10.2)
      DO 4002 I=1,N
      QGUT=(OVQ(I,ITYPL)+OVQ(I,ITYPR))*GLEN
 4002 QINL(I,IIN)=QINL(I,IIN)+QGUT
 4010 CONTINUE
      READ(5,3013) IROOF,IWRF,ROOFL
 3013 FORMAT(2I5,F5.2)
      GTIMO=0.00
      QINLO(IIN)=0.00000
      DO 4005 I=1,N
C     HERE QGUT SHOUS FLOW ENTERING THE INLET FROM DIRECT CONNECTIONS
      QGUT=IROOF*2.0*ROOFL*OVQ(I,IWRF)
      QINL(I,IIN)=QINL(I,IIN)+QGUT
 4005 CONTINUE
 4001 CONTINUE
      DO 5890 NINI=1,6
      WRITE(6,5500)
 5500 FORMAT('1')
      N15=5*NINI-4
      N16=5*NINI
      WRITE(6,5555)(I,I=N15,N16)
 5555 FORMAT(5X,'TIME FROM   DISCHARGE  DISCHARGE  DISCHARGE  DISCHARGE
     1 DISCHARGE',/,5X,'RAIN START  ENTERING   ENTERING   ENTERING   ENT
     2ERING   ENTERING',/,5X,'    IN       INLET ',I2,'   INLET ',I2,'
     3INLET ',I2,'   INLET ',I2,'   INLET ',I2,/,6X,'MINUTES       IN  CFS
     4    IN  CFS    IN  CFS    IN  CFS    IN  CFS',//)
      IF(N16-NINL) 5800,5800,5810
 5810 N16=NINL
 5800 WRITE(6,3014) GTIMO,(QINLO(IIN),IIN=N15,N16)
 3014 FORMAT(6X,F7.2,4X,F8.5,4(3X,F8.5))
      DO 5850 I=1,N
      GTIM=I*DT
      WRITE(6,5820) GTIM,(QINL(I,J),J=N15,N16)
 5820 FORMAT(6X,F7.2,4X,F8.5,4(3X,F8.5))
 5850 CONTINUE
      IF(N16-NINL) 5890,5950,5950
 5890 CONTINUE
 5950 DDD=1.0
C
C     ******************************************************
C     PROGRAM TO TRANSFORM HYDROGRAPHS INTO ONE MIN.INCREMENTS
C     ******************************************************
C
C     THIS IN ORDER TO FACILITATE THE ADDITION OF THE HYDROGRAPHS
C     DURING THE ROUTING THROUGH THE LATERAL AND MAIN SEWERS
      KK=DT/DDD
      NN=KK*N
      DO 6901 IIN=1,NINL
      DO 6901  I=1,N
      JJ=N+2-I
      JJJ=N+1-I
      QINL(JJ,IIN)=QINL(JJJ,IIN)
 6901 CONTINUE
      DO 6900 IIN=1,NINL
 6900 QINL(1,IIN)=0.00000
      DO 6912 IIN=1,NINL
      II=0
      DO 6911 I=1,N
      DO 6911 K=1,KK
      II=II+1
      QINP(II,IIN)=QINL(I,IIN)+((2*K-1)*(DDD/(2.0*DT))*(QINL(I+1,IIN)-QI
     1NL(I,IIN)))
 6911 CONTINUE
 6912 CONTINUE
      DO 6922 NINI=1,6
      WRITE(6,6916)
 6916 FORMAT('1')
      N15=5*NINI-4
      N16=5*NINI
      WRITE(6,6917)(I,I=N15,N16)
 6917 FORMAT(5X,'TIME FROM   DISCHARGE  DISCHARGE  DISCHARGE  DISCHARGE
     1 DISCHARGE',/,5X,'RAIN START  ENTERING   ENTERING   ENTERING   ENT
     2ERING   ENTERING',/,5X,'    IN       INLET ',I2,'   INLET ',I2,'
     3INLET ',I2,'   INLET ',I2,'   INLET ',I2,/,6X,'MINUTES       IN  CFS
     4    IN  CFS    IN  CFS    IN  CFS    IN  CFS',//)
      IF(N16-NINL) 6918,6918,6919
 6919 N16=NINL
 6918 DO 6920 I=1,NN
      TRTIM=DDD*(I-1)
      WRITE(6,6921) TRTIM,(QINP(I,J),J=N15,N16)
 6921 FORMAT(6X,F7.2,4X,F8.5,4(3X,F8.5))
 6920 CONTINUE
      IF(N16-NINL) 6922,6923,6923
 6922 CONTINUE
 6923 RETURN
      END
      SUBROUTINE PIROU
C
C     ***************************************
C     ROUTING THROUGH LATERAL AND MAIN SEWERS
C     ***************************************
C
C     USE OF MANNING'S EQUATION TO FIND THE AVERAGE TIME NEEDED
C     FOR A DISCHARGE HYDROGRAPH TO TRAVEL DOWNSTREAM UNTIL A
C     POINT(INLET,MANHOLE)WHERE A SECOND DISCHARGE HYDROGRAPH
C     ENTERS THE SEWER SYSTEM.ADD THE SHIFFTED AND THE NEW
C     HYDROGRAPH AND CONTINUE UNTIL THE POINT FOR WHICH WE SEEK
C     THE OUTFLOW HYDROGRAPH
C     'AREA' IS THE AREA OF FLOW IN SQUARE FEET
C     'SLOP' IS THE SLOPE OF THE INVERT IN FT/FT
C     'ALEN' IS THE LENGTH BETWEEN TWO CRITICAL POINTS(INLETS,
C     MANHOLES,CHANGE IN SLOPE OR DIAMETER OR SHAPE)
C     (WHEN THERE IS A CHANGE IN SLOPE OR DIAMETER OR SHAPE WE
C     CONSIDER THERE AN HYPOTHETICAL MANHOLE WITH ZERO INFLOW)
C     'PIPN' IS THE MANNING'S ROUGHNESS COEFFICIENT OF THE PIPE
C     'Y' IS THE DEPTH OF FLOW IN THE CONDUIT IN FEET
C     'HR' IS THE HYDRAULIC RADIUS IN FEET
C     'DIA' IS THE DIAMETER OF THE SEWER PIPE IN FEET OR ITS WIDTH
C     IF IT IS RECTANGULAR
C     'H'  IS THE HEIGHT OF RECTANGULAR SEWER IN FEET
C     I4=1 INDICATES CIRCULAR PIPE,I4=2 RECTANGULAR CONDUIT
C     NINL=NUMBER OF INLETS,MANHOLES AND CHANGES IN SLOPE AND DIAMETER
C     NPIPE=NUMBER OF PIPES
      COMMON    RI(70),AF(70),S(70),RM(70),AFM(70),P(70),D(70),VS(70),SS
     1(70),PP(70),SSIM(70),PPIM(70),VSIM(70),DIM(70),DATE(8),DC(70),DE(7
     20),OVQ(250,30),OVQW(70),NEW(30),QINL(70,20),QINLO(20),IW(250),V(25
     30),QLEAV(250),N,DT,NN,NINL,QINP(250,20)
      READ(5,909) NPIPE
  909 FORMAT(I5)
      NIN=NINL-1
      DO 1050 IIN=1,NIN
      J=IIN
      WRITE(6,917)
  917 FORMAT('1')
      READ(5,907) I4,SLOP,PIPN,DIA,H,ALEN
  907 FORMAT(I5,F10.6,F10.4,2F5.3,F10.3)
      KK=DT/1.0
      NN=KK*N
      IF(I4-1) 950,905,950
  905 QMAX=(0.46417*(DIA**2.6666)*(SLOP**0.5))/PIPN
      DO 931 I=1,NN
      IF(QINP(I,IIN)) 912,912,913
  912 V(I)=0.00
      IW(I)=0
      GO TO 931
  913 IF(QMAX-QINP(I,IIN)) 925,906,906
  925 IW(I)=1
      GO TO 931
  906 IW(I)=0
      B=(13.566*QINP(I,IIN)*PIPN)/((DIA**2.6666)*(SLOP**0.5))
      DO 930 K=1,6280,5
      SA=K
      X=SA/1000.0
      BB=((X-SIN(X))**1.6666)/(X**0.6666)
      IF(B-BB) 932,932,930
  930 CONTINUE
  932 V(I)=(8.0*QINP(I,IIN))/((DIA**2.0)*(X-SIN(X)))
  931 CONTINUE
      GO TO 970
  950 QMAX=(1.486*((DIA*H)**1.6666)*(SLOP**0.5))/(PIPN*((2.0*(DIA+H))**0
     1.6666))
      DO 960 I=1,NN
      IF(QINP(I,IIN)) 915,915,916
  915 V(I)=0.00
      IW(I)=0
      GO TO 960
  916 IF(QMAX-QINP(I,IIN)) 955,951,951
  955 IW(I)=1
      GO TO 960
  951 IW(I)=0
      B=(QINP(I,IIN)*PIPN)/(1.486*(SLOP**0.5)*(DIA**1.6666))
      M=H*1C00.0
      DO 959 K=1,M
      SA=K
      Y=SA/1000.0
      BB=(Y**1.6666)/((DIA+(2.0*Y))**0.6666)
      IF(B-BB) 953,953,959
  959 CONTINUE
  953 V(I)=QINP(I,IIN)/(DIA*Y)
  960 CONTINUE
  970 SUM=0.0
      SE=0.0
      DO 1000 I=1,NN
      SUM=SUM+(V(I)*QINP(I,IIN))
      SE=SE+QINP(I,IIN)
 1000 CONTINUE
      AVERV=SUM/SE
      AVT=ALEN/(AVERV*60.0)
      IAVT=AVT
      DIF=AVT-IAVT
      IF(DIF-0.5) 991,992,992
  991 AVERT=IAVT
      GO TO 993
  992 AVERT=IAVT+1
  993 IAVET=AVERT
      DO 994 I=1,IAVET
  994 QLEAV(I)=0.000
      IIAVT=IAVET+1
      NNEW=NN+IAVET
      NEEW=NN+1
      DO 995 I=IIAVT,NNEW
      KL=I-IAVET
  995 QLEAV(I)=QINP(KL,IIN)
      DO 996 I=NEEW,NNEW
      QINP(I,IIN)=0.000
      IW(I)=0
  996 V(I)=0.00
```

```
      WRITE(6,997) J
  997 FORMAT(28X,'*****************',/,28X,'PIPE  NUMBER  ',I2,/,28X,'***
     1**************',//)
      IF(I4-1) 968,969,968
  969 WRITE(6,967) DIA
  967 FORMAT(5X,'CIRCULAR CONDUIT  OF  DIAMETER   (FEET)
     1  ',F5.2)
      GO TO 962
  968 WRITE(6,966) DIA,H
  966 FORMAT(5X,'RECTANGULAR  CONDUIT  WITH  DIMENSIONS  IN FEET    ',F5
     1.2,'*',F5.2)
  962 WRITE(6,965) SLOP
  965 FORMAT(5X,'THE SLOPE OF THE PIPE INVERT IN FT/FT  IS
     1',F7.5)
      WRITE(6,964) ALEN
  964 FORMAT(5X,'THE LENGTH   OF  THE  PIPE  IN  FEET  IS
     1',F7.2)
      WRITE(6,963) PIPN
  963 FORMAT(5X,'THE  MANNINGS  ROUGHNESS  COEFFICIENT  IS
     1',F7.5)
      WRITE(6,998)QMAX
  998 FORMAT(5X,'MAXIMUM CAPACITY OF THE CONDUIT IS (CU.FEET/SEC)
     1 ',F6.2)
      WRITE(6,999) AVERV
  999 FORMAT(5X,'AVERAGE VELOCITY OF ENTERING DISCHARGES (FT/SEC)
     1  ',F5.2)
      WRITE(6,1001) AVERT
 1001 FORMAT(5X,'AVERAGE TIME OF TRAVEL OF THE ENTERING HYDROGRAPH(MIN)
     1  ',F5.2,//)
      WRITE(6,1002)
 1002 FORMAT(5X,'TIME FROM      DISCHARGE      VELOCITY WITH WHICH    DIS
     1CHARGE',/,5X,'RAIN START    ENTERING TO    ENTERING DISCHARGE    L
     2EAVING THE',/,5X,'(MINUTES)    THE PIPE(CFS)   TRAVELS DOWNSTREAM
     3   PIPE (CFS)',/,39X,'(FT/SEC)',/)
      IF(IIN-NIN) 1069,1078,1078
 1078 DO 1070 I=NEEW,NNEW
      QINP(I,NINL)=0.000
 1070 V(I)=0.00
      DO 1074 I=1,NNEW
 1074 QLEAV(I)=QLEAV(I)+QINP(I,NINL)
 1069 DO 1003 I=1,NNEW
      TIMP=(I-1)*1.0
      IF(IW(I)-1) 1072,1080,1072
 1072 WRITE(6,1004) TIMP,QINP(I,IIN),V(I),QLEAV(I)
 1004 FORMAT(6X,F7.2,7X,F8.5,13X,F5.2,11X,F8.5)
      GO TO 1003
 1080 WRITE(6,1005) TIMP,QINP(I,IIN)
 1005 FORMAT(6X,F7.2,7X,F8.5,7X,'CONDUIT OVERFLOWS')
 1003 CONTINUE
      DO 1020 I=NEEW,NNEW
 1020 QINP(I,IIN+1)=0.000
      DO 1021 I=1,NNEW
 1021 QINP(I,IIN+1)=QINP(I,IIN+1)+QLEAV(I)
 1050 CONTINUE
      RETURN
      END
```

Comments on the Developed Computer Program

1. The Program does not take into account the dry weather flow. If information and measurements are available from the urban area under consideration, the dry weather flow simply can be taken as an additional input for the routing through the sewers.

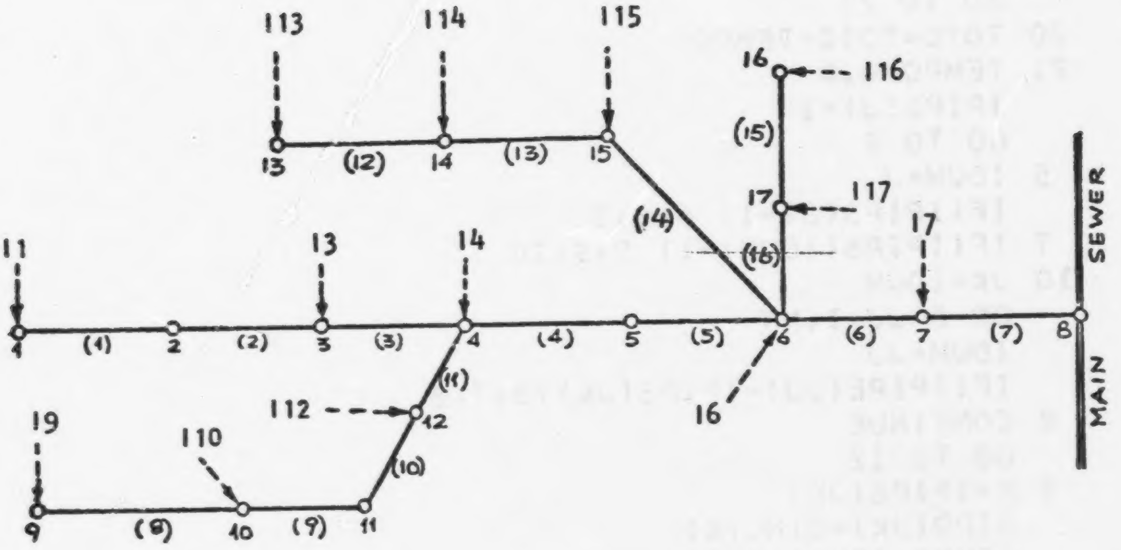
2. The SUBROUTINE subprogram "PIROU" has been written based on the assumption that only one lateral sewer drains the catchment under consideration. This is usually true for catchments of about 10 acres area.

If a broader subdivision of the whole basin will be preferred, then the catchments probably will be drained from a network of lateral sewers. In cases like this the SUBROUTINE "PIROU" has to be replaced by the program proposed on the next page, which deals with a pipe network. This program will use SUBROUTINE "PIROU" in the place of the statements 6, 9, 12, and 15. In Figure 152 a complex pipe network is presented with constant inflows at the inlets. Figure 152 shows the way of numbering the inlets and the pipes, necessary for the computer input.

Computer Program Dealing with a Pipe Network

```
      DIMENSION QINL(100),IPIPS(100),IPIPE(100),PIPQ(100)
C     NUMB.OF DOWNSTREAM LAST POINT OF MAIN BRANCH=MAIN
C     N NUMBER OF INLETS OR CHANGES IN SLOPE,SHAPE
C     NN  NUMBER OF PIPES
      READ(5,50)N,NN,MAIN
   50 FORMAT(3I5)
      READ(5,51)(QINL(I),I=1,N)
   51 FORMAT(12F5.2)
      READ(5,3)(IPIPS(J),IPIPE(J),J=1,NN)
    3 FORMAT(2I5)
      WRITE(6,150)
  150 FORMAT(10X,'DISCHARGE AT PIPE NUMBER&',/)
      TOTQ=0.0
      TEMPQ=0.0
      DO 1 I=1,MAIN
      DO 2 J=1,NN
      IF(IPIPE(J)-I) 2,5,2
    6 K=IPIPS(J)
      PIPQ(J)=QINL(K)
      TEMPQ=PIPQ(J)
   11 TOTQ=TOTQ+TEMPQ
      WRITE(6,100)TOTQ,J
  100 FORMAT(10X,F9.2,10X,I3)
      GO TO 21
   20 TOTQ=TOTQ+TEMPQ
   21 TEMPQ=0.0
      IPIPS(J)=1
      GO TO 2
    5 IDUM=J
      IF(IPIPS(J)-1) 6,6,7
    7 IF(IPIPS(IDUM)-1) 9,9,10
   10 JK=IDUM
      DO 8 JJ=1,NN
      IDUM=JJ
      IF(IPIPE(JJ)-IPIPS(JK))8,7,8
    8 CONTINUE
      GO TO 12
    9 K=IPIPS(JK)
      PIPQ(JK)=QINL(K)
      TEMPQ=TEMPQ+PIPQ(JK)
      IPIPS(JK)=1
      IF(IPIPE(JK)-I)10,11,10
   12 K=IPIPS(JK)
      PIPQ(JK)=QINL(K)
      TEMPQ=TEMPQ+PIPQ(JK)
      WRITE(6,102)TEMPQ,JK
  102 FORMAT(10X,F9.2,10X,I3)
      IF(IPIPE(JK)-I) 13,2,13
   13 DO 14 JJ=1,NN
```

```
      IF(IPIPS(JJ)-IPIPE(JK)) 14,15,14
   15 K=IPIPS(JJ)
      PIPQ(JJ)=QINL(K)
      TEMPQ=TEMPQ+PIPQ(JJ)
      WRITE(6,103)TEMPQ,JJ
  103 FORMAT(10X,F9.2,10X,I3)
      JK=JJ
      IF(IPIPE(JK)-I) 13,20,13
   14 CONTINUE
    2 CONTINUE
    1 CONTINUE
      CALL EXIT
      END
```


113
114
115
16
116
13
(12)
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(13)
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(15)
17
117
(14)
11
13
14
17
SEWER
(16)
1
(1)
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(2)
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(3)
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(4)
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(5)
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(6)
7
(7)
8
MAIN
(11)
112
12
16
19
110
(10)
9
(8)
10
(9)
11

Figure 152. COMPLEX PIPE NETWORK WITH CONSTANT INFLOWS AT THE INLETS

APPENDIX IV

DATA CARDS PREPARATION

A. Rainfall Data Cards

I. card 1 (8A4, F5.2, I5)

		Variable Names	Example Values
col. 1-20	Date of storm (day, month, year)	DATE	24 SEPT 1970
col. 21-32	Hour of start of storm (hours, min.,AM or PM)	DATE	5:35 AM
col. 33-37	Time interval between intensity values	DT	5.00
col. 38-42	Number of time steps to be calculated	N	36

II. card 2 (F5.2)

col. 1-5	First value of rainfall intensity	RO	0.20

III. Rainfall intensity cards, each (12F5.2)

	Intensity in in/hr	RI	

B. Infiltration Data Cards

I. card 1 (3F10.5, 2F8.4)

col. 1-10	Const. infiltr. rate (in/hr)	FC	0.53
col. 11-20	Initial infiltr. rate (in/hr)	FO	3.00
col. 21-30	Decay rate of infil. (1/min)	CK	0.0697
col. 31-38	Retention storage of pervious areas (in)	SD	0.250
col. 39-46	Retention storage of impervious areas (in)	SDIM	0.0625

	Variable Names	Example Values
II. card 2 (F5.2)		
col. 1-5 A number to divide DT in smaller intervals (see statement 19. If DT = 5.0 min, a logical value of AN is 20.0 or 30.0. Then the new intervals will be of 0.17 min. each)	AN	30.0

C. Overland Flow Data Cards

	Variable Names	Example Values
I. card 1 (I5)		
col. 1-5 Number of subcatchments to be simulated	NW	6
II. One card for each subcatchment (2I2, 3F7.4)		
col. 1-2 Number of the subcatchment	IA	1
col. 3-4 Indication number (1 for pervious, 2 for impervious)	INDW	1
col. 5-11 Length of area at the direction of flow (feet)	ALW	16.0
col.12-18 Manning's roughness coef.	ARW	0.35
col.19-25 Slope of the area at the direction of flow (ft/ft)	SW	0.01

D. Gutters Data Cards

	Variable Names	Example Values
I. card 1 (I5)		
col. 1-5 Number of entry points into the sewer system	NINL	9

(Data cards of type II, III, IV, V to be repeated for each entry point)

		Variable Names	Example Values
II. card 2 (I5)			
col. 1-5	Number of gutter branches leading into the entry point	NBRAN	2
(Data cards of type III, IV to be repeated for each gutter branch)			
III. card 3 (2I5)			
col. 1-5	Number of the first upstream gutter segment of the gutter branch	N1	1
col. 6-10	Number of the last downstream gutter segment of the gutter branch	N2	2
(Data cards of type IV to be repeated for each gutter segment)			
IV. card 4 (2I5, F10.2)			
col. 1-5	Number of the subcatchment providing flow to the gutter segment from the left	ITYPL	3
col. 6-10	Number of the subcatchment providing flow to the gutter segment from the right	ITYPR	5
col. 11-20	Length of the gutter segment in feet	GLEN	297.0
V. card 5 (2I5, F5.2)			
col. 1-5	Number of roofs directly connected with entry point	IROOF	0
col. 6-10	Number of subcatchment corresponding to the roof type	IWRF	6
col. 11-15	Length of the roofs (ft)	ROOFL	30.0

E. Pipes Data Cards

		Variable Names	Example Values
I. card 1 (I5)			
col. 1-5	Number of pipes	NPIPE	8
II. one card for each pipe (I5, F10.6, F10.4, 2F5.3, F10.3)			
col. 1-5	Indication number, 1 for circular pipe, 2 for rectangular	I4	1
col. 6-15	Slope of the pipe invert	SLOP	0.0045
col. 16-25	Manning's roughness coefficient	PIPN	0.013
col. 26-30	Diameter of circular pipe or width of rectangular pipe (ft)	DIA	3.0
col. 31-35	Height of rectangular pipe (0.0 for circular) (ft)	H	0.0
col. 36-45	Length of pipe (ft)	ALEN	120.0

APPENDIX V

DATA CARDS FOR THE FIRST CASE STUDY

```
 24 SEPTFMBER 1970      5 35  AM  5.0   36
0.20
0.20 0.20 0.70 0.20 0.20 0.20 0.42 0.42 0.50 0.62 0.87 1.63
5.30 5.25 2.80 1.75 1.25 0.88 0.80 0.67 0.50 0.47 0.47 0.47
0.35 0.25 0.25 0.25 0.25 0.25 0.25 0.25 0.25 0.25 0.25 0.25
     0.53       3.00      0.0697  0.2500  0.0625
30.0
   6
1 1   16.0   0.35   0.01
2 1   40.0   0.35   0.01
3 1   70.0   0.35   0.01
4 2   17.0  0.013   0.02
5 2    8.0  0.013 0.0375
6 2   12.0  0.012  0.667
   9
   2
   1    2
   3    5       297.0
   1    4       140.0
   3    4
   5    3       297.0
   4    1       140.0
   0    6 30.0
   4
   5    5
   2    4       105.0
   6    6
   4    2       105.0
   7    7
   4    2       105.0
   8    8
   2    4       105.0
  12    6 30.0
   4
   9    9
   2    4       105.0
  10   10
   4    2       105.0
  11   11
   4    2       105.0
  12   12
   2    4       105.0
  12    6 30.0
   4
  13   13
   2    4       105.0
  14   14
   4    2       105.0
  15   15
   4    2       105.0
  16   16
   2    4       105.0
  12    6 30.0
   4
  17   18
 5    3        297.0
 4    1        140.0
19   20
 3    5        297.0
 1    4        140.0
21   22
 3    5        297.0
 1    4        140.0
23   24
 5    3        297.0
 4    1        140.0
 0    6  30.0
 4
25   25
 2    4        105.0
26   26
 4    2        105.0
27   27
 4    2        105.0
28   28
 2    4        105.0
12    6  30.0
 4
29   29
 2    4        105.0
30   30
 4    2        105.0
31   31
 4    2        105.0
32   32
 2    4        105.0
12    6  30.0
 4
33   33
 2    4        105.0
34   34
 4    2        105.0
35   35
 4    2        105.0
36   36
 2    4        105.0
12    6  30.0
 2
37   38
 5    3        297.0
 4    1        140.0
39   40
 3    5        297.0
 1    4        140.0
 0    6  30.0
 8
 1    0.0045      0.013  3.0  0.0       120.0
 1    0.0045      0.013  3.0  0.0       210.0
 1    0.0045      0.013  3.0  0.0       210.0
 1    0.0045      0.013  3.0  0.0       120.0
 1    0.0045      0.013  3.0  0.0       120.0
 1    0.0045      0.013  3.0  0.0       210.0
 1    0.0045      0.013  3.0  0.0       210.0
 1    0.0045      0.013  3.0  0.0       120.0
```

MATHEMATICAL MODEL FOR SIMULATION OF RUNOFF QUALITY FROM AN URBAN WATERSHED

1. INTRODUCTION

This study is devoted to the development of a mathematical formula for the concentration of pollutants at any location within a combined sewer system of an urban watershed. A stochastic process is used to describe the removal of surface pollutants by the runoff water and inertial effects of solids during motion are taken into account. The pollutants are routed from the inlets through the lateral and main sewers.

2. PREVIOUS WORK

A lot of information concerning urban runoff pollution is available, but only a few studies have included data on both runoff quantity and quality in sufficient detail so that the quality of direct surface runoff can be evaluated as a function of runoff intensities.

A recent study under the sponsorship of the Environmental Protection Agency, Water Quality Office by a consortium of contractors--Metcalf & Eddy, Inc., the University of Florida, and Water Resources Engineers, Inc. [9] --provides probably the most comprehensive runoff quality model currently available. The model presented here, as developed at the University of Cincinnati, has similarities with the above referenced model; however, a major difference is that an integral solution has been developed instead of a stepwise solution.

3. DEVELOPMENT OF THE MATHEMATICAL MODEL FOR URBAN RUNOFF QUALITY

Wastewater, surface dust and dirt, catch basin trappings, and air pollutant residues are the major sources of storm runoff pollution.

The pollutants in this study are classified as soluble and non-soluble. The following are soluble: BOD, COD, total hydrolyzable phosphates, various forms of nitrogen, and various forms of coliforms; while suspended solids are recognized as non-soluble.

3.1 Mathematical Formula for Surface Pollutant Removal

Surface pollutants consist of street litter and dustfalls that appear on the surface at the start of an oncoming storm.

The following assumptions are made as the basis for the development of the mathematical formulation:

a. Decaying effects of pollutants due to chemical changes or biochemical degradation during the running-off period are neglected.

b. The amounts of pollutants percolating into the soil by infiltration are neglected.

c. The rate of removal of pollutants by runoff water is assumed to be proportional to the amount of pollutant remaining, and to the runoff intensity.

d. Because the distribution of air pollutants in the atmosphere is non-uniform, spatially and unsteady, varying from time to time, it is difficult to formulate the amount which runs off in rainfall; therefore, this source has not been evaluated separately but is considered to be included in the overall runoff constituents.

The amount of a pollutant remaining on a runoff surface at a particular time, the rate of runoff at that time, and the general characteristics of the watershed can be related in a simplified form as follows:

$$\frac{-dP}{dt} = KqP \tag{1}$$

Rearranging,

$$\frac{-dP}{P} = Kqdt$$

Integrating,

$$\int_{P_o}^{P} \frac{dP}{P} = -\int_{0}^{t} Kqdt = -K\int_{0}^{t} qdt$$

$$\ln \frac{P}{P_o} = - K \int_0^t qdt = - K V_t$$

where $$V_t = \int_0^t qdt$$

$$\frac{P}{P_o} = e^{-KV_t}$$

$$P = P_o e^{-KV_t} \qquad (2)$$

In equations (1) and (2),

P_o = amount of pollutant on the surface drainage area at the start of an oncoming storm

P = amount of pollutant remaining on the surface at time t

q = runoff intensity at time t

V_t = accumulated runoff water volume up to time t

e = the base of natural logarithms

K = a constant characterizing the drainage area.

Thus, the remaining amount of pollutant at any instant on the surface is assumed to be decreasing exponentially as a function of the accumulated runoff volume.

Probabilistically, $P/P_o = e^{-KV_t}$ can be considered as the probability that a polluting particle on the surface will still remain at its original position when storm runoff water volume has accumulated to V_t at time t.

Consequently,

$$\phi = 1 - \frac{P}{P_o} = 1 - e^{-KVt} \qquad (3)$$

where ϕ is the probability that a polluting particle will be removed from its original position after time t. Differentiating equation (3) with respect to V:

$$\frac{d\phi}{dV} = f(V) = K\ e^{-KVt} \qquad (4)$$

where f(V) is the probability density function for ϕ. The integral of f(V) over all possible values of V is unity:

$$\int_0^{\infty} f(V)\ dV = \int_0^{\infty} Ke^{-KV}\ dV = -e^{-KV}\Big]_0^{\infty} = 1$$

It is assumed that the function f(V) is nowhere negative, and is single-valued.

Therefore the probability that a particle is removed from its original position during the time from t_1 to t_2, with corresponding accumulated volume of runoff from V_1 to V_2, is

$$Pr(V_1 \leq V \leq V_2) = \int_{V_1}^{V_2} d\phi = \int_{V_1}^{V_2} Ke^{-KV} dV = -e^{-KV}\Big]_{V_1}^{V_2} = e^{-KV1} - e^{-KV2} \qquad (5)$$

Assuming that once a soluble pollutant is picked up by runoff water it will flow into an inlet,the amount of pollutants which are flushed into a sewer inlet is:

$$P_o\ (e^{-KV1} - e^{-KV2}) = P_o\ e^{-KV1} - P_o\ e^{-KV2} = P_1 - P_2$$

Figure 153 schematically represents the removal of soluble surface pollutants by runoff water.

For the transport of solids by runoff water, the drag force exerted by flowing water on a particle varies as the square of the flowing velocity, i.e.

$$F = C_d \frac{\rho \upsilon^2}{2} A_c$$

Hence, the portion of the solids that are removed from their original positions and flushed into a sewer inlet is assumed to be proportional to the square of the runoff intensity.

$$r = \lambda\ q^2 \qquad (6)$$

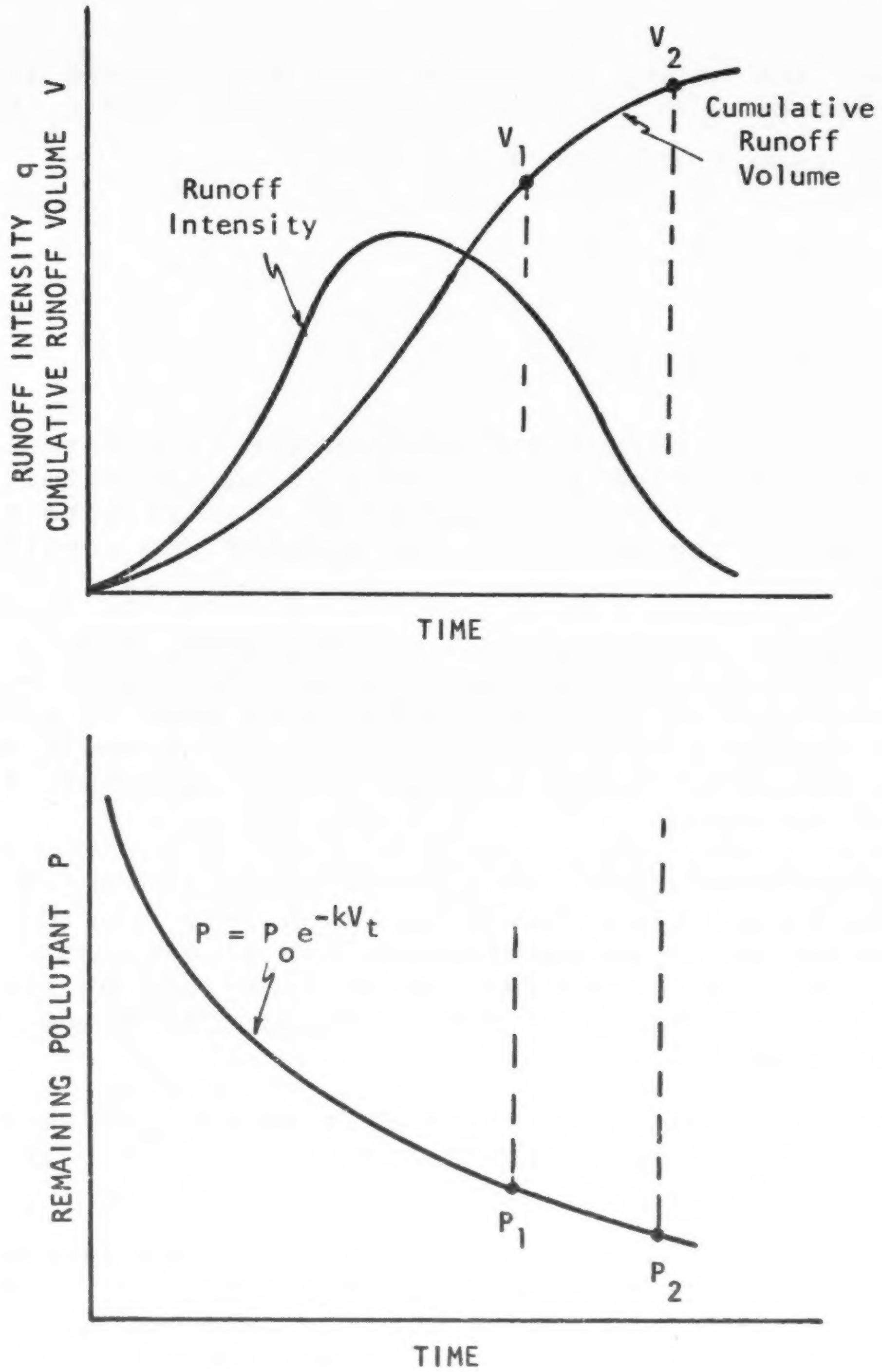

Figure 153. SCHEMATIC REPRESENTATION OF SURFACE POLLUTANT REMOVAL BY RUNOFF WATER

In equation (6), r is the fraction of the removed solids that is carried off into a sewer inlet at the instant when the runoff intensity is q; λ is a proportionality factor.

Thus, the amount of solids that are flushed into a sewer inlet during the same time interval from t_1 to t_2 with mean runoff intensity

$$\bar{q} = \frac{q_1 + q_2}{2}, \text{ is}$$

$$P_o \lambda \bar{q}^2 (e^{-KV1} - e^{-KV2}) \qquad (7)$$

Thus, with the previous assumptions and this formula, it is obvious that the closer a particle is to the inlet, the smaller the runoff rate necessary to wash it into the inlet, and the larger the particle, the greater the runoff rate required.

Metcalf & Eddy, Inc., et al, [9] found that a rainfall rate of 0.14 in/hr would begin to move 20 micron material, and a rate of 0.65 in/hr would begin to move 40 micron material. Also a runoff rate of 0.01 in/hr would transport 20 micron material and a rate of 0.03 in/hr would transport 40 micron material.

If the sewer inlet is a catch basin, then the retention of solids in the trap should be taken into consideration. In its relation to sedimentation, the size-weight composition of a suspension can be expressed by the frequency distribution of the settling velocities of constituent particles [35].

A hypothetical cumulative distribution of settling velocities of a suspension carried into a sewer inlet is presented in Figure 154.

Thus, with the size distribution of the flushed-in suspension and the flowing rate of runoff, υ_o, the portion of the flushed-in suspension that is retained in the catch basin can be estimated at any instant during the running-off period.

The proportion $1 - \theta_o$ of particles settling with velocities $\upsilon_s \leqslant \upsilon_o$ is retained in its entirety. The remainder

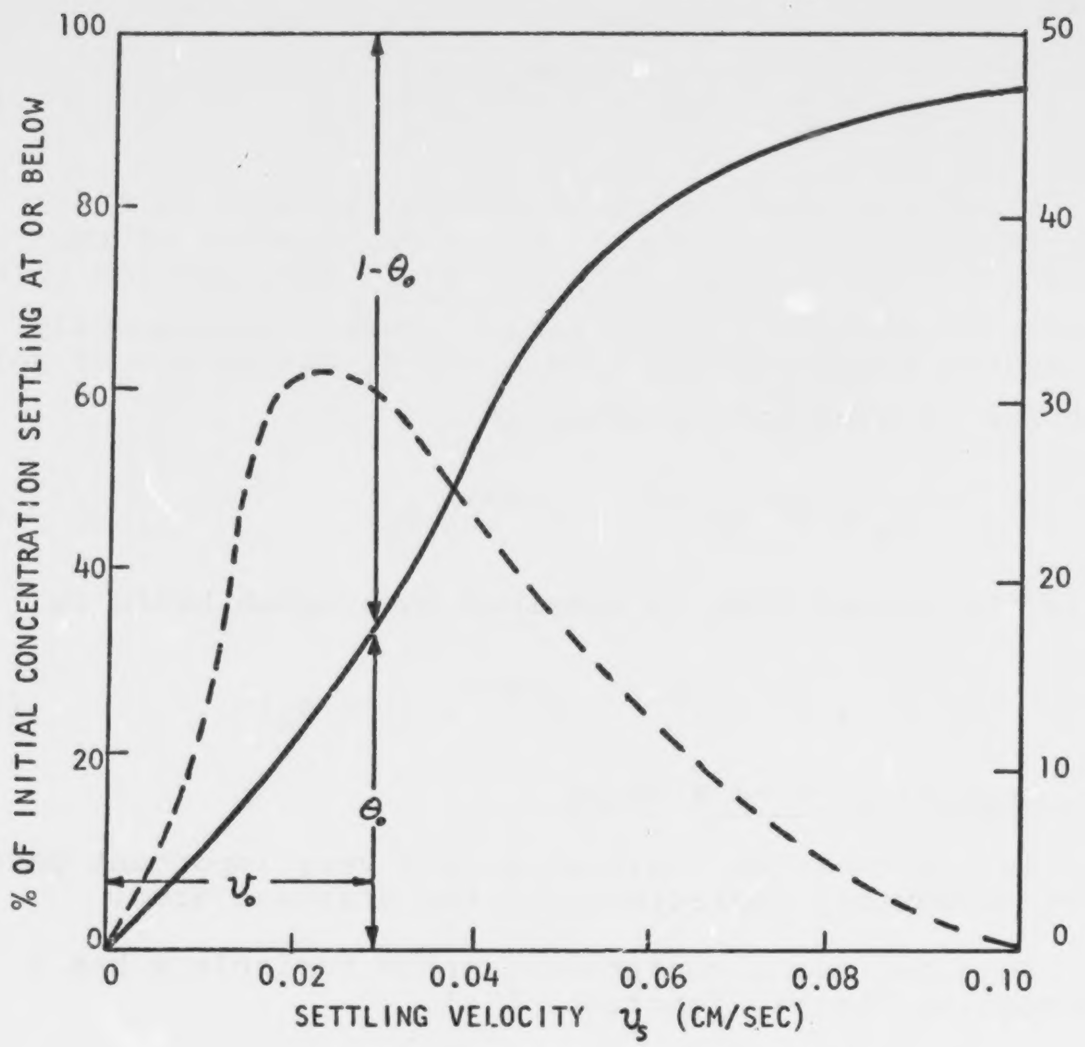

Figure 154. HYPOTHETICAL FREQUENCY DISTRIBUTION AND CUMULATIVE DISTRIBUTION OF SETTLING VELOCITIES OF A SUSPENSION

$$\int_0^{\theta_o} \frac{\upsilon_s}{\upsilon_o} d\theta$$

reaches the bottom, and the overall retention portion becomes

$$\psi = (1 - \theta_o) + \int_0^{\theta_o} \frac{\upsilon_s}{\upsilon_o} d\theta \tag{8}$$

Since the catch basin is generally short in the direction of flow, hence $1 - \theta_o$ is the portion of the suspension that is retained, and θ_o is that portion which is carried by runoff water into a sewer. Consequently, during the time interval from t_1 to t_2 the amount of solids that are flushed into a sewer is

$$P_o \lambda \bar{q}^2 (e^{-KV_1} - e^{-KV_2}) \theta_o \tag{9}$$

while the amount that is retained in a catch basin is

$$P_o \lambda \bar{q}^2 (e^{-KV_1} - e^{-KV_2}) (1 - \theta_o) \tag{10}$$

3.2 Significance of K Value

In all these derivations K is a very important parameter presumably characterizing the drainage area.

Suppose that a continuous random variable x has a probability density function f(x).

The expectation E(x) of the random variable x is defined as [36]:

$$E(x) = \int_{-\infty}^{\infty} x\, f(x)\, dx$$

which may be interpreted as the center of gravity of the probability density function f(x).

In this study, the probability density function is

$$f(V) = Ke^{-KV_t}$$

$$E(V) = \int_0^\infty V\ f(V)\ dV = \int_0^\infty V\ K\ e^{-KV}\ dV$$

$$d(VKe^{-KV}) = KVde^{-KV} + e^{-KV}\ d(KV) = -K^2\ Ve^{-KV}\ dV + Ke^{-KV}\ dV$$

$$KVe^{-KV}\ dV = \frac{1}{K}\left[Ke^{-KV}\ dV - d(KVe^{-KV})\right]$$

$$E(V) = \int_0^\infty KVe^{-KV}\ dV$$

$$= \frac{1}{K}\int_0^\infty\left[Ke^{-KV}\ dV - d(KVe^{-KV})\right]$$

$$= \frac{1}{K}\left\{\int_0^\infty Ke^{-KV}\ dV - KVe^{-KV}\Big]_0^\infty\right\}$$

$$= \frac{1}{K}\left\{-e^{-kv}\Big]_0^\infty - KVe^{-KV}\Big]_0^\infty\right\} = \frac{1}{K} = E(V) \qquad (11)$$

Statistically, E(V) is the expected value or mean value of the runoff water volume which will theoretically remove all the pollutants in the long run.

In other words, since $f(V) = Ke^{-KV}$, $f(0) = K$ is the probability density function at the start of a runoff. The greater the K value, the more pronounced the first flushing effect of a drainage area will be, because a smaller volume of runoff water is required to accomplish the same amount of pollutant removal from an area with a greater K value.

In order to illustrate the determination of the K value, assume 90% of the surface pollutants on a drainage area were found to be removed into the sewer after a runoff of 0.5 inch.

Thus, from

$$\phi = 1 - e^{-KV_t}$$

$$0.90 = 1 - e^{-0.5K}$$

$$e^{-0.5K} = 0.1$$

$$K = 4.6$$

The factors that will affect the K value presumably are:

(a) size and shape of a drainage area,

(b) slope of gutters,

(c) types of storm water inlets,

(d) time of concentration of a drainage area,

(e) kind of pollutant to be transported.

3.3 Removal of Pollutant from Catchbasins

Catchbasins historically have been constructed on inlets to combined sewers and storm drains for the purpose of removing heavy grit and detritus which might otherwise settle out in the piping system and providing a water seal to confine sewer gas.

The trapped pocket of liquid and solids in which organic materials undergo decomposition between storms contributes a substantial amount of pollutants to urban runoff water.

The American Public Works Association [37] determined the way the soluble pollutants in a catchbasin at the start of a storm are flushed into the sewer. They experimented by adding 15 to 45 pounds of sodium chloride dissolved in water to a catchbasin containing 353 gallons. Water from a hydrant was discharged through a hose and water meter to the gutter near the catchbasin. Samples were taken from the effluent when various quantities of water up to 1685 gallons had been added to and passed through the catchbasin.

Figure 155 presents the cumulative percent of salt discharged as a function of gallons of liquid added.

A formula regressioned by Metcalf and Eddy, Inc. [9] to fit the curve in Figure 155 came out to be

$$R = \left[1.0 - e^{-\frac{V}{1.6G}}\right] \cdot 100 \qquad (12)$$

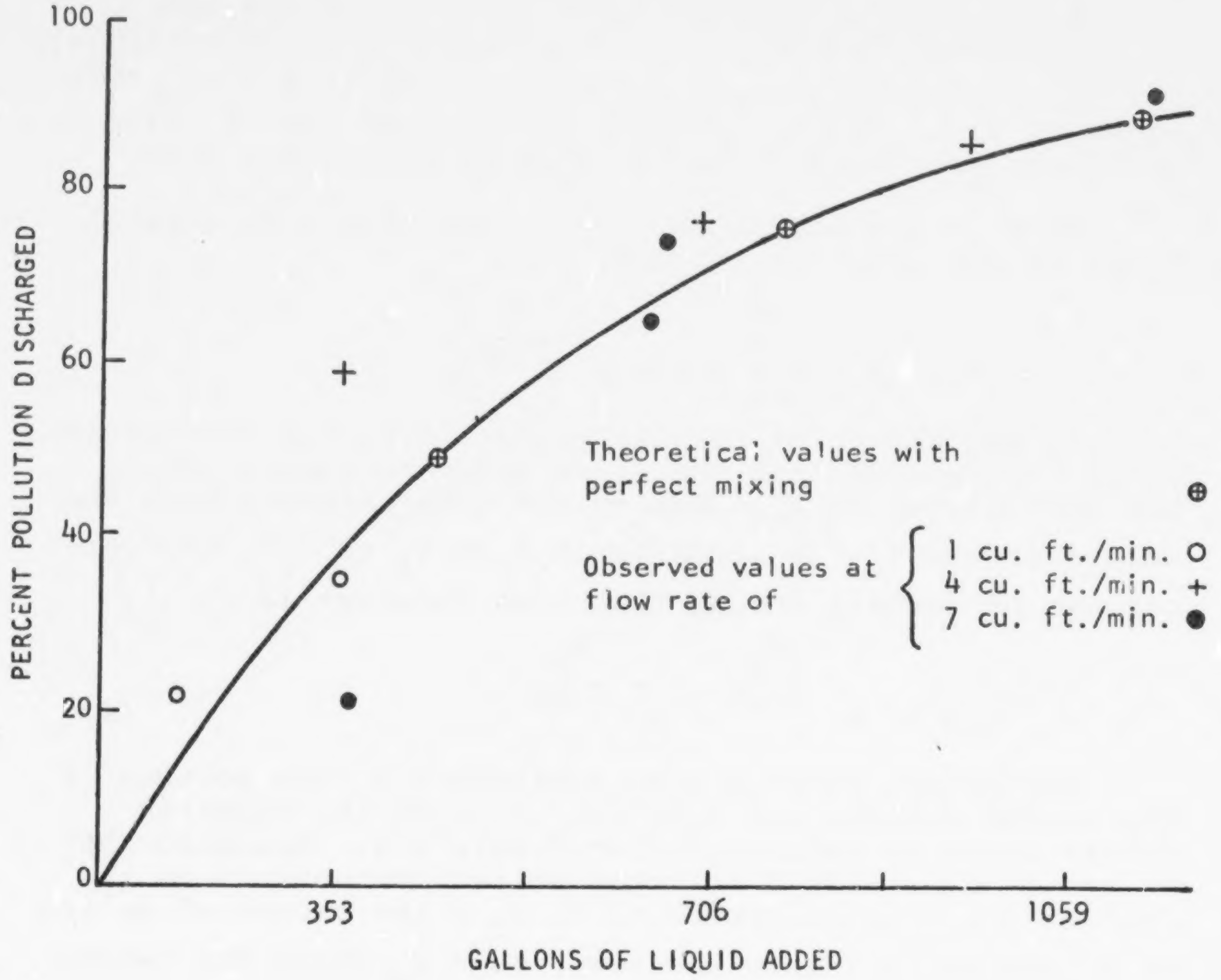

Figure 155. RELATIONSHIP OF FLOW INTO CATCHBASIN AND REDUCTION OF CONCENTRATION ON SALT (NaCl)

In equation (12),

R = percent of salt removed,

V = cumulative inflow volume to the catchbasin, and

[illegible] = trapped volume of liquid in a catchbasin.

A scrutiny of equation (12) reveals the similarity of this empirical equation and the equation derived previously for soluble pollutant removal in the form of $\phi = 1 - e^{-KV_t}$, equation (3). Hence, it might be inferred that K value is inversely proportional to the size of a drainage area.

If Y is the amount of soluble pollutants in a catchbasin at the start of a storm, then

$$Y \cdot R = Y \left[1.0 - e^{-\frac{V_t}{1.6G}} \right]$$

will be the amount of soluble pollutant in the catchbasin to be flushed out into the sewer when the runoff volume has accumulated to V_t. During the time interval when the volume of runoff has accumulated from V_1 to V_2, the percentage of removal during that time interval is

$$R = e^{-\frac{V_1}{1.6G}} - e^{-\frac{V_2}{1.6G}} \qquad (13)$$

For solids removal from a catchbasin, the portion of the solids flushed out into the sewer by the incoming runoff could be estimated from Figure 154. Assuming that an amount of solids Z is retained in a catchbasin at the instant the flushing velocity is υ_o, the amount of solids to be flushed out into the sewer is $Z\ \theta_o$ while the amount $Z\ (1 - \theta_o)$ is to be retained.

3.4 Routing of Soluble Pollutants through Lateral and Main Sewers

For simplicity in routing soluble pollutants through sewer systems, it is assumed that the decaying effects due to chemical and biochemical changes are negligible.

The soluble pollutant is assumed to travel at current velccities along the distance from where it enters the inlet to any point downstream in the sewer system.

Routing takes account of the volume of runoff water and amount of pollutant removed into an inlet from the farthest inlet area upstream.

At the next inlet point, the contributions from the next inlet area are added to the previously routed pollutant from upstream. This process is continued at successive points.

The total amount of soluble pollutant at the outlet end of the lateral sewer at the mean time from t_1 to t_2 can be expressed mathematically as

$$W = \sum_1^n x_i + \sum_1^n P_{oi}\left[e^{-KV_{t_1-t_r}} - e^{-KV_{t_2-t_r}}\right] + \sum_1^n Y_i\left[e^{-\frac{V_{t_1-t_r}}{1.6G}} - e^{-\frac{V_{t_2-t_r}}{1.6G}}\right] + \int_{t_1}^{t_2} c_s q_s dt \qquad (14)$$

The concentration of the routed soluble pollutant at the outlet end of the lateral sewer is

$$c = \frac{W}{\int_{t_1}^{t_2} Q dt + \int_{t_1}^{t_2} q_s dt} \qquad (15)$$

In equations (14) and (15)

$\sum_1^n x_i$ = estimated amount of pollutant due to rainfall contamination in the air, and n is the number of inlet contributions routed.

$$\sum_1^n P_{oi}\left[e^{-KV_{t_1-t_r}} - e^{-KV_{t_2-t_r}}\right] =$$

total amount of surface soluble pollutant flushed into inlets by runoff water, and P_{oi} denotes the amount of soluble pollutant on the surface drainage area served by the i-th inlet, and t_r is the travel time from the i-th inlet to the outlet end.

$$\sum_{1}^{n} Y_i \left[e^{-\frac{V_{t_1-t_r}}{1.6G}} - e^{-\frac{V_{t_2-t_r}}{1.6G}} \right] =$$

total amount of soluble pollutant flushed out from catch basins.

$\int_{t_1}^{t_2} c_s q_s dt$ = total amount of pollutants in dry weather flow from t_1 to t_2, and c_s is the concentration of the pollutant in dry weather flow, and q_s is the dry weather flow rate.

$\int_{t_1}^{t_2} q_s dt$ = dry weather flow volume monitored at the outlet end from t_1 to t_2.

$\int_{t_1}^{t_2} Q\, dt$ = runoff volume at the outlet end during the routing period from t_1 to t_2 .

Routing of soluble pollutants through the main sewer follows the same method as that used for the lateral sewers by taking the outlet pollutants of the laterals as the individual contributions to the main sewer.

Diagrammatical representation of the routing of soluble pollutants through a sewer system is shown in Figure 156.

3.5 Routing of Settleable Solids in Sewer Systems

Heavy solids in sanitary sewage water and in runoff water are swept down the sewer invert like the bed load of streams. Light materials float on the water surface.

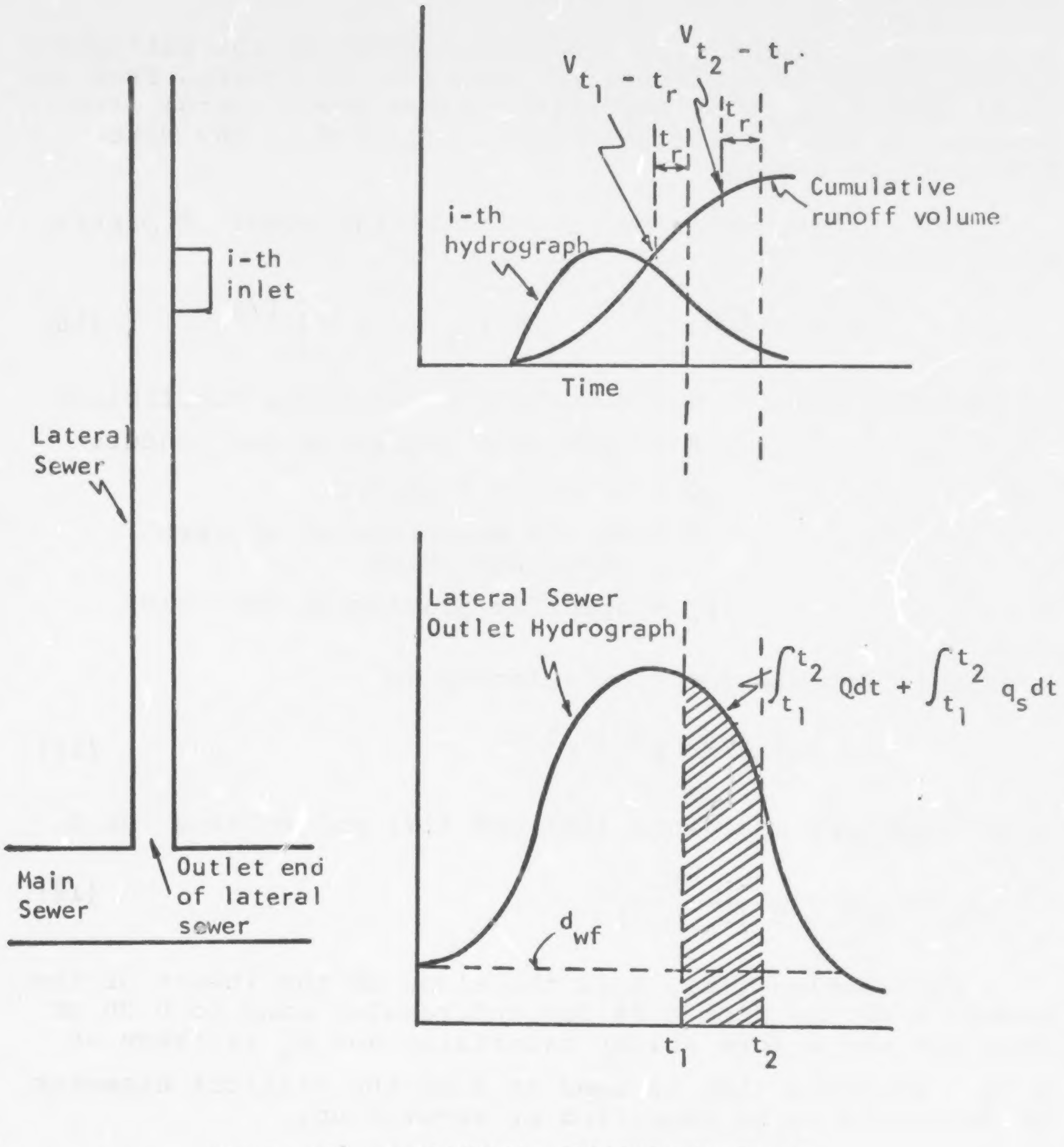
$V_{t_1} - t_r$
$V_{t_2} - t_r$
t_r
t_r
Cumulative runoff volume
i-th hydrograph
Time
i-th inlet
Lateral Sewer
Main Sewer
Outlet end of lateral sewer
Lateral Sewer Outlet Hydrograph
$\int_{t_1}^{t_2} Q dt + \int_{t_1}^{t_2} q_s dt$
d_{wf}
t_1
t_2

Figure 156. DIAGRAMMATIC REPRESENTATION OF SOLUBLE POLLUTANT ROUTING

When the velocity in the conduit falls, the heavy solids are left behind as bottom deposits; when velocities rise again, gritty substances are picked up once more and moved along in heavy concentrations.

This section of the study is devoted to the estimation of the amount of sediments in sewers to be scoured from the sewer bottom or to be deposited on the sewer bottom with respect to the varying velocities involved in the water transport in sewers.

The channel velocity [35] initiating scour of particles deposited in sewers is defined as

$$V = \frac{1.486}{n} R^{1/6} \left[B\ (s_s - 1)\ d \right]^{1/2} \tag{16}$$

In equation (16): n = Manning's n roughness coefficient

R = hydraulic radius of the conduit

d = particle diameter

B = shield magnitude of sediment characteristics

s_s = specific gravity of particles

Manning's formula for flow velocity is

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \tag{17}$$

After equating equations (16) and (17) and solving for d:

$$d = \frac{R\quad S}{B\ (s_s - 1)} \tag{18}$$

In equation (18), S is the slope of the invert of the sewer; B varies from 0.04 for unigranular sand to 0.06 or more for nonuniform sticky materials; and s_s is taken as 2.70. Equation (18) is used to find the critical diameter of sediments to be deposited or scoured up.

Case I. Sediment Uptake: that portion of sewer sediments that has particle sizes greater than or equal to the critical diameter found from equation (18) will remain at the bottom of the sewer. Those particles with diameters smaller than the critical diameter will be scoured up and transported downstream.

Case II. Sediment Deposition: that portion of the suspension in the incoming flows with particle diameters greater than or equal to the critical diameter found from equation (18) will settle to the bottom of the sewer. Those particles with diameters smaller than the critical diameter will be transported farther.

The sewer sediment characteristic is approximated by a plot prepared by Metcalf and Eddy, Inc. [9] and shown in Figure 157.

Routing of solids in sewer systems can be accomplished by applying the continuity equation of mass transport, starting from the farthermost upstream point of the sewer and moving downstream.

Routing procedures for settleable solids take the following steps:

(a) Calculate the average velocity in each conduit.

(b) Calculate the hydraulic radius in the conduit of concern.

(c) Determine the critical diameter of particles by equation (18).

(d) Determine the fraction of particles with diameters greater than that calculated in step (c).

(e) Determine sediment scour and deposition by considering the mass balance in the sewer conduit considered.

Figure 158 shows the contributing settleable suspension flow sources in a sewer conduit considered for routing of solids.

M_1 = amount of sediments at the sewer bottom at time t_1 ,

x_1 = fraction of M_1 with particle diameters greater than or equal to the critical diameter calculated in equation (18),

M_2 = amount sediments at the sewer bottom at time t_2 ,

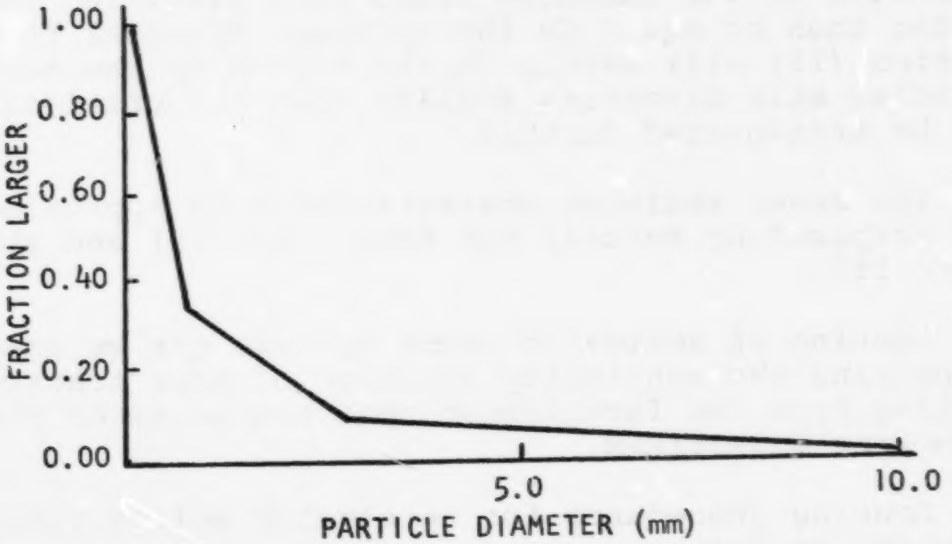

Figure 157. SIEVE ANALYSIS PLOT FOR SEWER SEDIMENT (AFTER METCALF & EDDY, INC.)

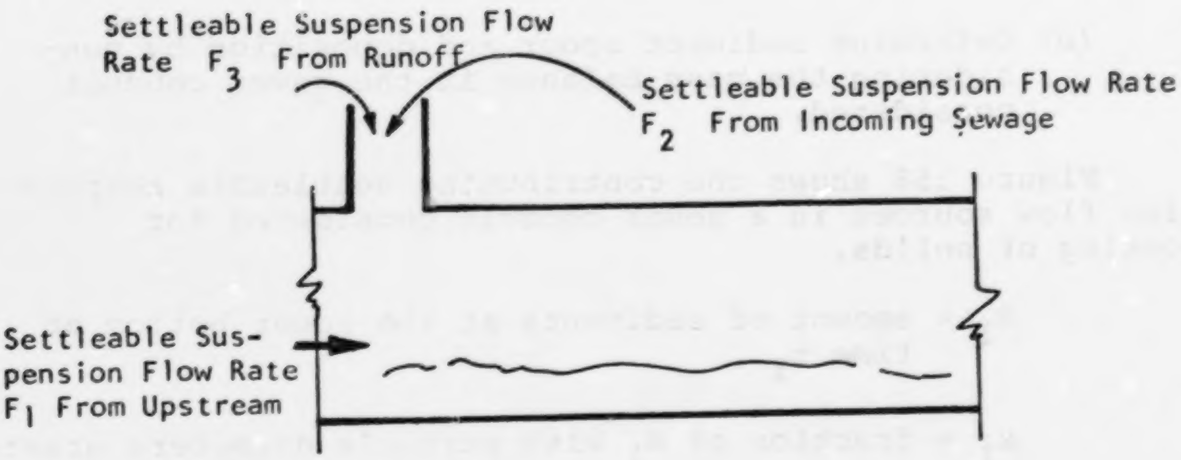
Settleable Suspension Flow Rate F3 From Runoff
Settleable Suspension Flow Rate F2 From Incoming Sewage
Settleable Suspension Flow Rate F1 From Upstream

Figure 158. MASS FLOW IN A SEWER CONDUIT

$\Delta t = t_2 - t_1$

F_1 = settleable suspension flow rate from sewer conduit immediately upstream,

F_2 = settleable suspension flow rate of incoming sewage,

F_3 = settleable suspension flow rate of incoming runoff,

x_2 = fraction of settleable suspension flows with particle diameters greater than the critical diameter calculated in equation (18),

V_o = current volume of wastewater in the sewer conduit under consideration,

Q_o = incoming flow rate at the sewer conduit considered.

Then the total amount of sediment at the sewer conduit under consideration at time t_2 is

$$M_2 = M_1 + x_2\ (F_1 + F_2 + F_3)\ \Delta t \tag{19}$$

The concentration of the suspension which will flow into the next sewer conduit is

$$\text{Concentration} = \frac{(1.0 - x_1)\ M_2}{V_o} + \frac{(1.0 - x_2)\ (F_1 + F_2 + F_3)}{Q_o} \tag{20}$$

4. APPLICATION OF THE MODEL

At the present stage, equation (15) is tested by considering the drainage area as a whole, instead of being made up of a number of individual inlet drainage areas with individual inlet hydrographs which are routed through the system and combined with other inlet areas as the routing proceeds downstream. Contribution of the pollutants from the air was neglected, and catchbasin contributions were included in surface loadings. Dry weather flow was assumed to have a constant rate and a constant level of quality during the whole period of storm runoff. Furthermore, in these tests the K value is chosen as 4.6, based upon the assumption that after a runoff of 1/2 inch, 90% of the surface pollutants are removed from the whole drainage area.

The Laguna Street combined sewer draining a steep drainage area of 370 acres in San Francisco [9] was selected as a demonstration site for the verification of the model. One storm occurred there on March 10, 1967 yielding a total runoff of 0.22 inch, and another storm occurred 5 days later with a total runoff of 0.33 inch. Both storms were analyzed assuming a dry weather flow of 1.5 cfs with a 250 mg/l BOD concentration. The surface loading of BOD at the start of the storm was estimated to be 3.85 pounds/acre. Tables 65 and 66 tabulate the computations, and Figures 159 and 160 show the computed results and the measured values. The results reveal a good agreement of the trends of the variation of BOD concentrations during the whole period of runoff.

The "Bloody Run" combined sewer watershed of approximately 2380 acres in Cincinnati, Ohio, was also selected to test the model. A storm occurred there on April 1, 1970 with a total runoff of 0.10 inch. The dry weather flow was at 1.5 cfs with BOD at a level of 300 mg/l. The surface BOD loading was estimated to be 5.65 pounds/acre. The results of computations and trends of variations of BOD concentrations are shown in Table 67 and Figure 161.

San Francisco Laguna Street
March 10, 1967 Storm

TABLE 65

Time	q in/hr	ΔV in	V in	D $e^{-KV_1} - e^{-KV_2}$	$\frac{P_o \cdot D}{\Delta V \cdot 83.8}$	Diluted Sewage BOD	Result BOD mg/l
20:35			0.0				
	0.02	0.01		0.04496	76.1	41.6	117.7
21:05			0.01				
	0.02	0.005		0.02172	74.0	41.6	115.6
21:20			0.015				
	0.03	0.0075		0.03165	72.0	29.4	101.4
21:35			0.0225				
	0.045	0.01125		0.04547	68.5	20.4	88.9
21:50			0.03375				
	0.055	0.014		0.05340	64.8	17.0	81.8
22:05			0.04775				
	0.02	0.01		0.03609	61.5	41.6	103.1
22:35			0.05775				
	0.025	0.00625		0.02173	59.0	29.4	88.4
22:50			0.064				
	0.03	0.0075		0.02526	57.3	7.5	64.8
23:05			0.0715				
	0.13	0.0325		0.09994	52.3	4.7	57.0
23:20			0.1040				
	0.21	0.0525		0.13297	43.0	7.5	50.0
23:35			0.1560				
	0.13	0.0325		0.0676	37.4	4.7	42.1
23:50			0.1890				
	0.08	0.02		0.03685	31.4	11.9	43.3
24:05			0.2090				
	0.04	0.013		0.01719	22.5	22.7	45.2
24:25			0.2190				

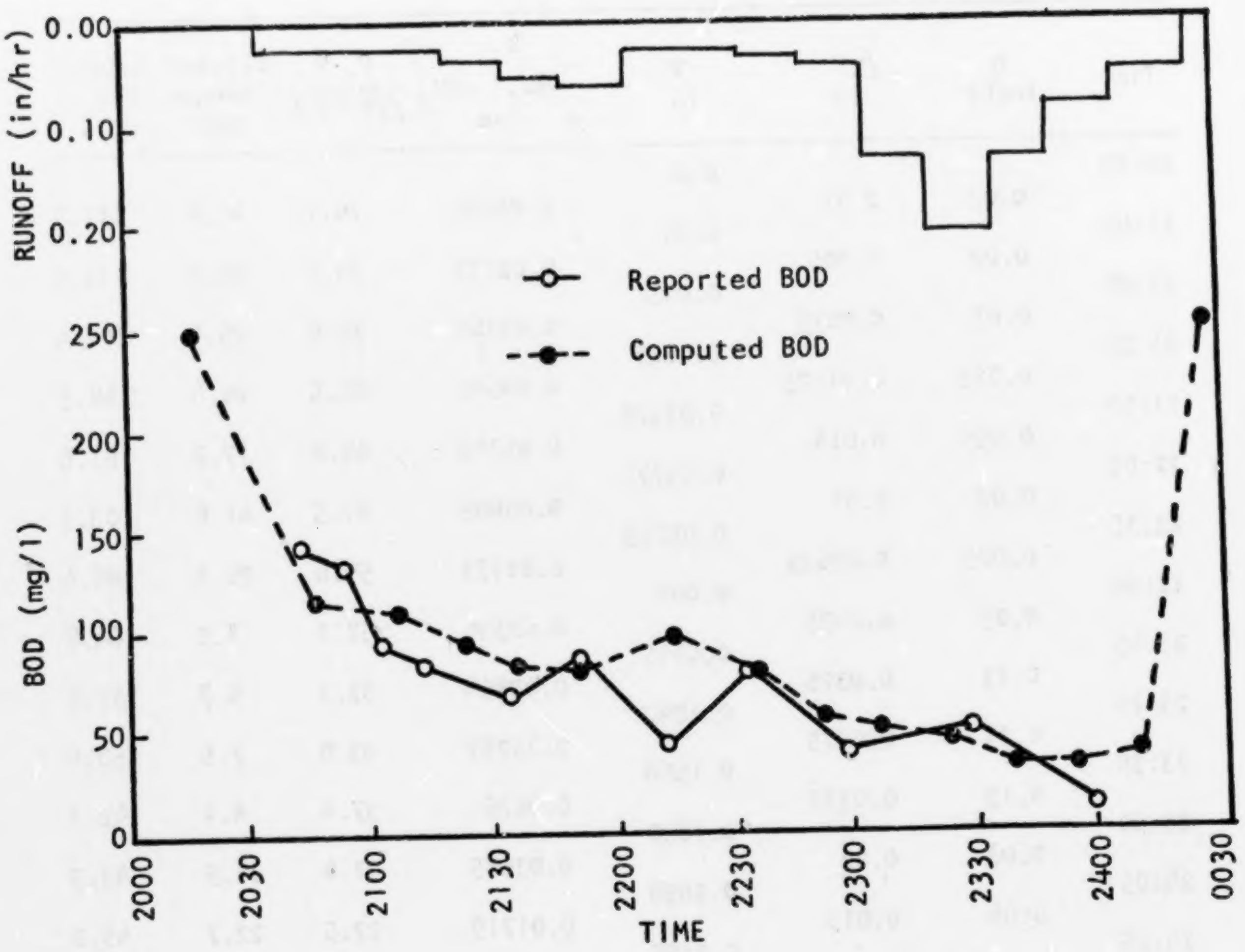

Figure 159. SAN FRANCISCO LAGUNA STREET COMBINED SEWER
March 10, 1967 Storm

San Francisco Laguna Street
March 15, 1967 Storm

TABLE 66

Time	q in/hr	ΔV in	V in	D $e^{-KV_1} - e^{-KV_2}$	$\frac{P_o \cdot D}{\Delta V * 83.8}$	Diluted Sewage BOD	Result BOD mg/l
20:15			0.0				
	0.02	0.005		0.02274	77.4	41.6	119.0
20:30			0.005				
	0.03	0.0075		0.03314	75.0	29.4	104.4
20:45			0.0125				
	0.025	0.00625		0.02676	73.0	34.5	107.5
21:00			0.01875				
	0.015	0.00375		0.01569	71.2	52.6	123.8
21:15			0.0225				
	0.01	0.0025		0.01031	70.0	71.4	141.4
21:30			0.025				
	0.005	0.00125		0.00511	69.5	111.0	180.5
21:45			0.02625				
	0.0	0.0		0.0	0.0	250.0	250.0
22:00			0.02625				
	0.06	0.015		0.05909	67.2	15.6	82.8
22:15			0.04125				
	0.205	0.05125		0.17372	57.5	4.8	62.3
22:30			0.0925				
	0.21	0.0525		0.14020	45.5	4.7	50.2
22:45			0.1450				
	0.08	0.02		0.04511	38.4	11.9	50.3
23:00			0.1650				
	0.12	0.03		0.06034	34.2	8.05	42.3
23:15			0.1950				
	0.19	0.0475		0.08004	28.7	5.15	33.9
23:30			0.2425				
	0.18	0.045		0.06128	23.1	5.43	28.5
23:45			0.2875				
	0.15	0.0375		0.04222	19.2	6.5	25.7
24:00			0.3250				

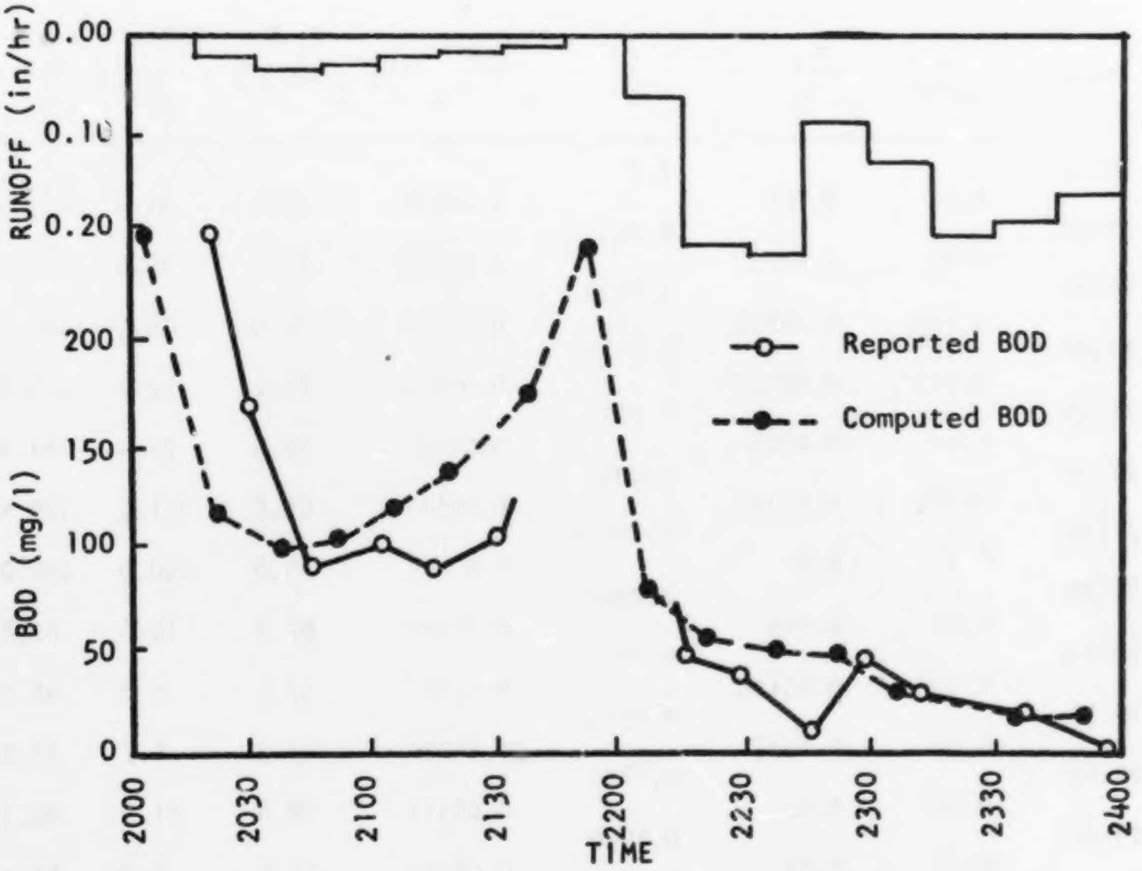

Figure 160. SAN FRANCISCO LAGUNA STREET COMBINED SEWER
Storm of March 15, 1967

Cincinnati Bloody Sewer
April 1, 1970 Storm

TABLE 67

Time	q in	ΔV in	V in	D $e^{-KV_1}-e^{-KV_2}$	$\frac{P_o \cdot D}{\Delta V * 57.0}$	Diluted Sewage BOD	Result BOD mg/l
5:05			0.0				
	0.06	0.0075		0.03391	113.0	62.6	175.6
5:12.5			0.0075				
	0.107	0.013375		0.05765	108.0	35.0	143.0
5:20			0.020875				
	0.123	0.015375		0.06203	101.0	30.5	131.5
5:27.5			0.036250				
	0.118	0.01475		0.05552	94.0	31.8	125.8
5:30			0.05100				
	0.102	0.01275		0.04505	88.5	36.8	125.3
5:37.5			0.06375				
	0.087	0.010875		0.03639	83.7	42.2	125.9
5:50			0.074625				
	0.074	0.009375		0.02994	80.0	50.8	130.8
5:57.5			0.084				
	0.056	0.007		0.02153	77.0	67.2	144.2
6:05			0.091				
	0.034	0.00425		0.01274	75.0	110.0	185.0
6:12.5			0.09525				
	0.026	0.00325		0.00957	74.0	145.0	219.0
6:20			0.09850				
	0.022	0.00275		0.00799	72.6	170.0	242.6
6:27.5			0.10125				

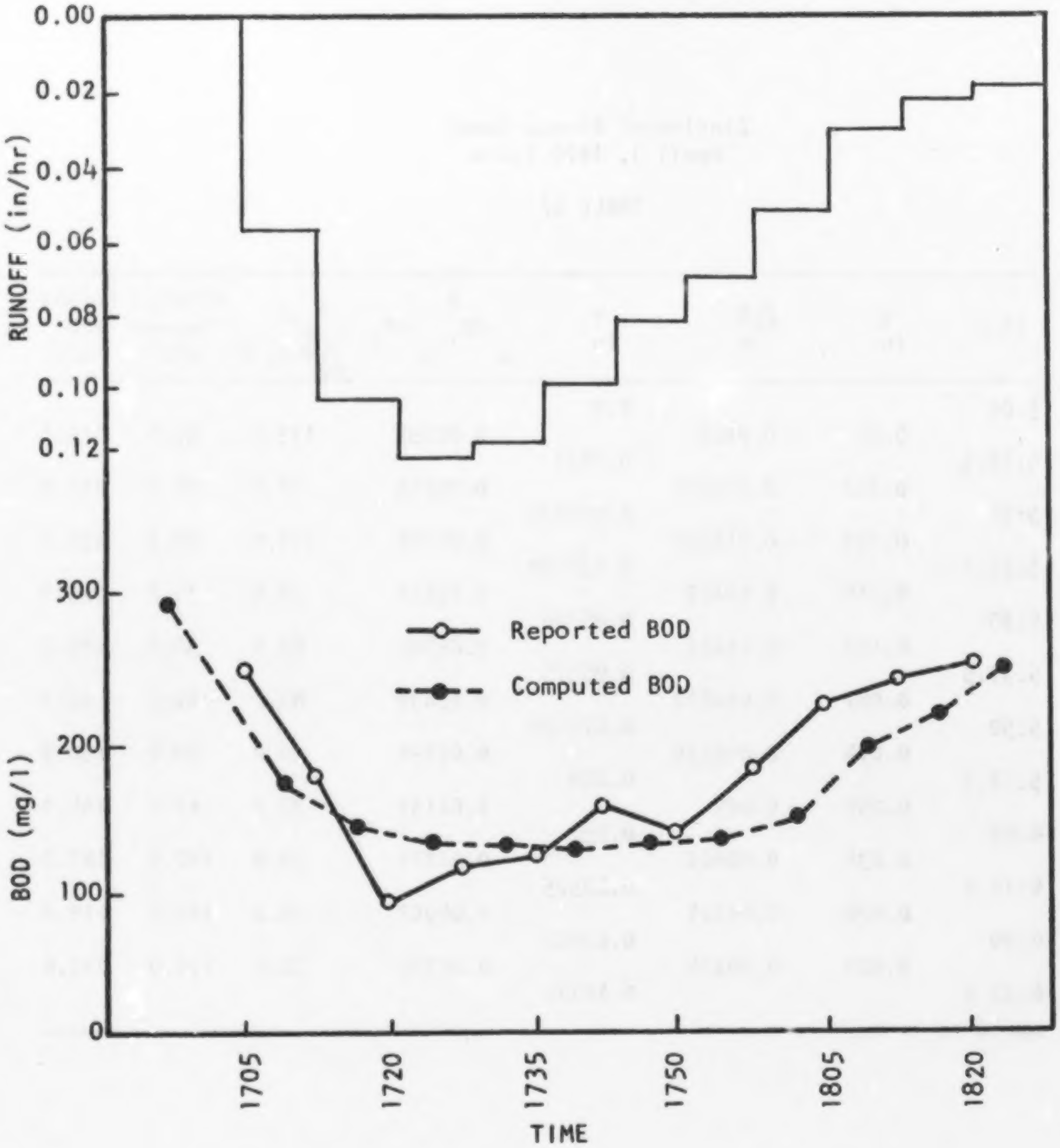

Figure 161. CINCINNATI BLOODY RUN SEWER
Storm of April 1, 1970

5. CONCLUSIONS AND SUGGESTIONS FOR FURTHER STUDY

Conclusions

1. The quality of urban runoff can be modeled from urban runoff hydrographs and pollutant contributions from the associated environments.

2. The amount of surface pollutants in an inlet drainage area removed during the time that the volume of runoff water increases from V_1 to V_2 is expressed by

$$P_o\left(e^{-KV_1} - e^{-KV_2}\right)$$

and correspondingly the amount of soluble pollutants in a catchbasin is indicated by

$$Y\left(e^{-\frac{V_1}{1.6G}} - e^{-\frac{V_2}{1.6G}}\right)$$

3. The concentration of soluble pollutants at any monitoring station in the drainage area can be found from equation (15) after finding W from equation (14). Equation (15) is repeated below.

$$c = \frac{W}{\int_{t_1}^{t_2} Q dt + \int_{t_1}^{t_2} q_s dt}$$

Suggestions for Further Study

The present study is confined to theoretical considerations of pollutant removal and transport in the drainage system. Verification or modification is anticipated as field data become available.

A determination of the surface pollutant removal constant K as a function of the physical characteristics of various drainage areas would be worthwhile research.

Experimental determination on the proportionality factor λ in equation (7) for the removal of solids into an inlet by runoff water would make the formula for solid removal more reliable.

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SECTION VIII

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This is an interim report on investigations for the development of a comprehensive storm water management model

Detailed information on the watershed characteristics and data on runoff quantity and quality have been compiled from a one year study of a combined sewer watershed of approximately 2380 acres in Cincinnati, Ohio. Collection of these data is planned to continue over a several year period. The information collected will be used to test and develop practical storm water management models.

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